In this Issue: EXPANSION JOINTS in Concrete Structures.—Cleveland's Municipal GARBAGE REDUCTION Works.—Concrete Towers for ELECTRIC TRANSMISSION LINES.—The ERUPTION of Mauna Loa.

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A Journal of Civil, Mechanical, Mining and Electrical Engineering.

Volume 57. No. 18. | 220 Broadway, NEW YORK, THURSDAY, MAY 2, 1907. | $5.00 per Year. Single Copies, 15 Cents.

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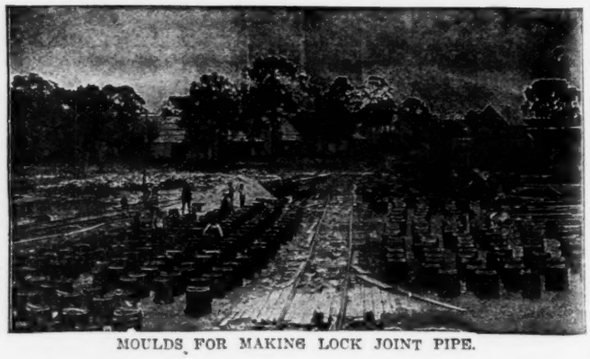

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A JOURNAL OF CIVIL, MECHANICAL, MINING AND ELECTRICAL ENGINEERING

Vol. 57. No. 18. 220 Broadway, New York, May 2, 1907.

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THE GREAT LAVA FLOW OF 1907 FROM THE FLANKS OF MAUNA LOA, HAWAII.

By E. D. BALDWIN.*

On Wednesday night, Jan. 9, at midnight, a very brilliant reflection suddenly lit up the whole top of Mauna Loa, showing that great fountains of lava were spouting up in the summit crater of Mokaweoweo, which is at an elevation of 13,675 ft. above the sea. A good idea of the brightness of this reflection as seen from Hilo, 38 miles distant from the summit crater, is afforded by the statement made to the writer by a gentleman who passed through the San Francisco earthquake and fire, that the reflection was as intense and brilliant as that of the great San Francisco fire.

This brilliant reflection diminished toward morning, and was hardly visible for a few nights following, as the lava seemed to be working its way down through the interior of the mountain, which was evidenced by the numerous earthquake shocks, over 50 light shocks being felt in the Kau district, two of which were felt at Hilo.

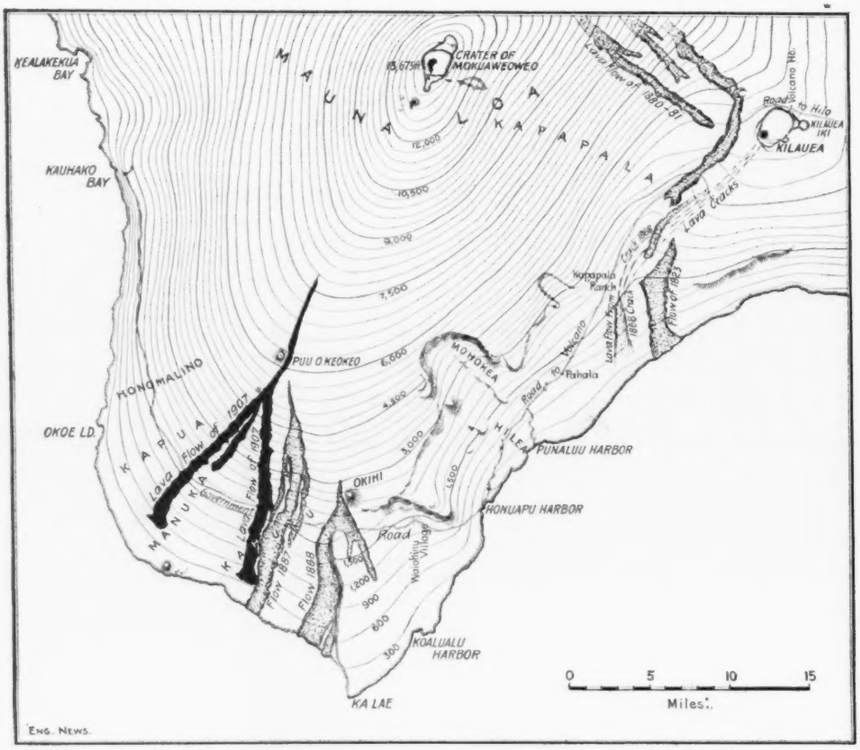
Some time during Saturday night, Jan. 13, the lava flow broke out high up on the southern slope of Mauna Loa, at an elevation of between 7,000 and 8,000 ft., and some fifteen miles from the summit. The lava flowed with great rapidity down the steeper sides of the mountain, crossing the Government Road about 9 a. m. the next morning, as the telephone communcation was cut off about that time. This branch of the flow reached a point about one mile from the sea in four days, and then piled up and spread out on the sides, and stopped flowing the following Saturday, Jan. 19. The north branch of the flow crossed the Government Road on Tuesday, Jan. 15, about noon, and came within a few miles of the sea; the movement of this branch stopped about a week after. The small branch between the north and south branches was very active during Sunday and Monday, Jan. 20 and 21, but the whole action in the flow stopped a few days after, followed by a great renewal of activity at the crater of Kilauea, which has kept up very steadily since.

One of the most interesting occurrences on the breaking out of a great lava flow from Mauna Loa is the rush of the people to see the wonderful phenomenon; one would naturally think that every one was fleeing from the flow, but, on the contrary, every available horse and conveyance is engaged, and people rush to the flow. On Sunday night, Jan. 14, only 24 hours after the lava broke out, over 150 people reached the south branch and saw the little branch flow or tongue across the road, close to the 1887 flow. On the Kona side at the north branch, over 200 people had arrived in all sorts of conveyances and automobiles in time to see this branch cross the Government Road.

Both of the lava flows crossing the Government Road are of the massive "aa" formation, of a light reddish blue gray color, and are about one-half mile wide, with an average depth of 15 ft., and about five miles apart. "Aa" lava is the rough granulated rock, of all sizes from a fraction of an inch in diameter up to great boulders of ten to twenty feet in diameter, piled to a height of 12 to 40 ft. The outside granules of these rocks are somewhat brittle, but the interior is generally very compact, tough and hard. Nearly all of the lava from Mauna Loa flows in the form of aa at first and later, if the flow keeps up for several months, it changes to the "pahoehoe," or smooth rock form. The aa or granulated form is caused by the greater stirring activity and more moisture in the lava at first, and when the action quiets down it seems to pour out from the same source in the pahoehoe form. The lava probably follows the same course as melted iron, which will granulate if water is poured on it at certain temperatures.

VIEWS OF THE LAVA FLOW OF 1907 FROM THE VOLCANO OF MAUNALOA, ISLAND OF HAWAII.

*Baldwin & Alexander, Civil Engineers, Judd Building, Honolulu, Hawaii.



MAP OF PART OF THE ISLAND OF HAWAII, SHOWING LAVA FLOW OF 1907 FROM THE SOUTHERN SLOPE OF MAUNA LOA.

An aa flow in motion down the steep slopes of the mountain is a grand sight, and one beyond description. The lava seems to move forward in great belches in a liquid form, with great boulders floating down the stream, and as it cools the supply of lava flowing in underneath presses up the rocks into great heaps, and then a new belch will break out from beneath.

The flow of 1907 has covered about 9,000 acres of rough, rocky land, of no value, and in volume it has poured out over 200,000,000 cu. yds. of new rock.

Some comparisons of this late lava flow with some of the former great lava flows from Mauna Loa may be of interest at this time. The great lava flow of 1855 broke out on the Hilo side of the mountain at an altitude of 10,500 ft., and flowed 13 months, a length of 25 miles, of an average width of one mile. This flow ejected in the neighborhood of 600,000,000 cu. yds. of rock, and covered over 15,000 acres. The source of the lava flow of 1880-81 was near that of the 1855 flow; its duration was nine months, and with its three great branches it ejected over 540,000,000 cu. yds. of rock, which covered over 20,000 acres of land. The great eruption of 1859 reached the sea in eight days, flowing a distance of 33 miles, and ejected probably as much material as the 1855 flow. The lava flow of 1907 lasted only two weeks, yet in that short time it ejected one-third as much material as that of the great 1855 flow, which lasted thirteen months.

The accompanying photographs show views of the north branch. They were taken by Honolulu photographers, who landed from the steamers on the rough coast, with a large crowd of sightseers, and tramped up a mile or so to the foot of the flow. The map has been platted by the writer from government maps and the 1907 flow is located with the aid of a few bearings and elevations which he took on the spot.

THE GOLD OUTPUT OF THE TRANSVAAL MINES in February is given as 494,623 oz., valued at $19,505,000. The gold production of South Africa reached a maximum in December, 1906, of 541,953 oz., which was an increase of 33.3% over the pre-war record of July, 1899. The February output is a reduction of 10.6% from the December record, due partially to the fact that February is a short month.

EXPANSION JOINTS IN CONCRETE STRUCTURES WITH SPECIAL REFERENCE TO BLOCK CONSTRUCTION IN DRYDOCKS AND RESERVOIRS.

By L. F. BELLINGER,* M. Am. Soc. C. E.

Although it is well known that concrete expands and contracts under a change of temperature with a given coefficient of expansion and also shrinks as it hardens or sets in air with an as yet undetermined coefficient of shrinkage, yet the exact amount that a certain body of concrete will change its dimensions under a given change of temperature and time of setting is not easily computed. The effects of temperature changes and shrinkages in setting, as exhibited in ragged, unsightly cracks and leakage of water through same, the disintegration of the structure through action of frost in, and rusting of steel reinforcement exposed by these cracks, are well known. The methods of providing for the stresses which produce these effects are not so well known, and the structural details providing for these stresses are in many instances inadequate, or wholly lacking.

In the effort to prevent the cracking caused by thermal expansion and shrinkage due to setting, two methods are usually employed. The concrete is either built as a monolith with reinforcing steel to take up the stresses induced by the change in form of the concrete and thus prevent their transference to the weaker material, or sliding joints, which allow a movement in the main body of the structure, are placed at regular intervals along the work.

An approximate method of computing the amount of steel necessary to take up the stresses in the concrete can be computed under the following assumptions, taken in the main from Buel and Hill's "Reinforced Concrete" but with the constants in use by the Bureau of Yards and Docks, U. S. Navy Department.

THERMAL AND SHRINKAGE STRESSES.

Concrete is a poor conductor of heat and in large bodies the average temperature of the whole mass rises and falls very slowly; so slowly in fact, that for retaining walls, dams, locks, etc., it is probable that the mean monthly temperatures can be taken as the temperatures of the whole mass. Therefore the range of temperature affecting the expansion and contraction of concrete masonry is that from the mean monthly summer temperature, to the mean monthly winter temperature, or, to be safer, the range is that from the maximum monthly mean of summer to the minimum monthly mean of winter, as shown in the following table:

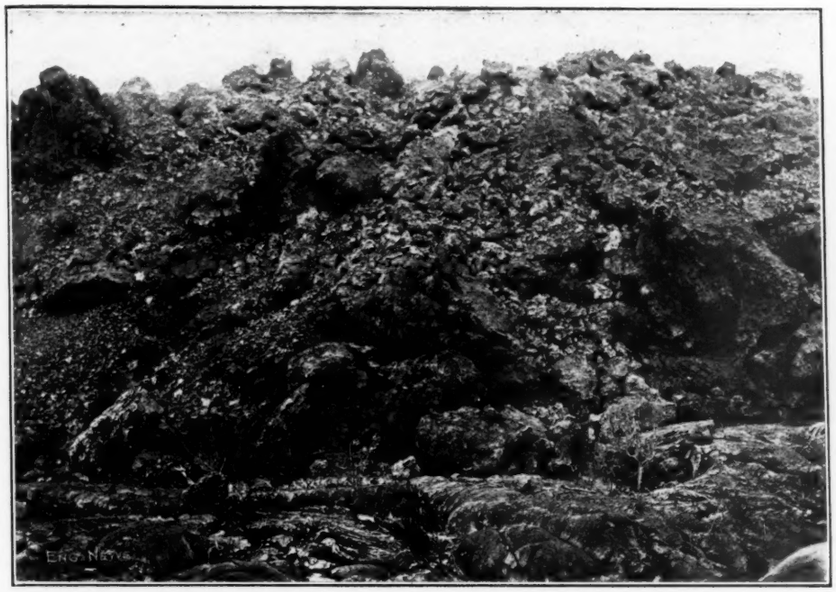
TEMPERATURE RANGE IN THE UNITED STATES.

	Mean summer to mean winter.	Maximum monthly mean to minimum monthly mean.
New England	37.7°	41.4°
Middle Atlantic States	41.9°	42.3°
West Gulf States	30.7°	35.3°
Middle Pacific Coast Region	15.8°	17.7°

This table used with the coefficient of expansion for one degree F., easily remembered as "point.

*Civil Engineer, U. S. Navy, New York Navy Yard.

FOOT OF THE NORTH BRANCH OF 1907 LAVA FLOW, 35 FT. HIGH.



VIEW SHOWING STRUCTURE OF LAVA FLOW.

five noughts, five, five" (.0000055) will provide data for temperature stress computations in heavy concrete work.

The shrinkage of concrete which sets in air may be partly accounted for as a contraction caused by fall in temperature. Cement in setting or crystallizing rises rapidly in temperature; 12-in. cubes of neat Portland cement have reached 212°F. and Portland concrete cubes of same size 85° to 90° F. The interior of a large concrete mass, like a dam or dock, may reach 100°F. Furthermore, if concrete is mixed on a board, spread out thin, and allowed to take its initial set, then remixed within 1½ to 2 hours, when placed next to old concrete, it will not shrink away from the old work as fresh concrete does. The cause of this is probably due partly to air cooling of the thin layer on the board, and partly to the cooling effect of the additional water required in remixing. This shrinkage is rapid during the first 28 days, but in large concrete masses, continues for years. In the Boonton (N. J.) dam which had 55% of cyclopean rubble stone (1 yd. to 2½ yds. in size) embedded in the concrete, records indicate that the concrete cooled very slowly and contraction cracks continued to increase in width for several years. Since concrete is much like cast-iron in some respects, it is probable that a mass of concrete, like a large casting, sets first on the surface, and as the interior cools down, contraction keeps increasing until actual compression in some of the surface particles exists.

Various experiments show that while concrete which hardens in water swells .0002 to .0005 parts, concrete, hardening in air, shrinks .0003 to .0005 parts, an average of .0004 parts per unit length. This means a ½-in. crack in a wall 100 ft. long in addition to contraction caused by any atmospheric drop in temperature. In Brightwood reservoir, Washington, D. C., the 50 ft. sections placed alternately, showed shrinkage cracks about ¼ in. wide. Air hardened concrete swells when immersed in water, and water hardened concrete shrinks when exposed to the air. The shrinkage of .0004 parts per unit length will produce tensile stresses in the concrete unless the concrete is free to contract. Movement of unreinforced walls being prevented by friction on the foundation and surrounding material and structural connections, the walls act as if fixed at the ends, and when the shrinkage stresses exceed the tensile strength of the concrete, such walls crack at varying intervals. The greater the friction on surrounding media, the shorter will be the intervals between cracks; the stronger the concrete, the larger will be these intervals. Cracks occur as in a chain, at the weakest point, which is where there is the greatest freedom of motion or where the strength of the wall is a minimum. Since the reinforcement in a wall of given length is fixed at the ends in the concrete, its length cannot change. Therefore the summation of stresses in the steel caused by shrinkage of concrete must be zero. Then, where the concrete is weakest, the steel will be in tension, and where the concrete is strongest, the steel will be in compression, the external friction being assumed uniform.

The maximum compressive stress in the steel will be its change in length, times the modulus of elasticity, $.0004 \times 30{,}000{,}000 = 12{,}000$ lbs. per sq. in. The maximum tensile stress that can come upon steel from shrinkage of concrete is the maximum tensile strength of the concrete, 300 lbs. per sq. in., and the percentage of steel required for shrinkage alone is $300 \div 37000 = 0.81\%$; or for 50,000 lbs. elastic limit $= 300 \div 50{,}000 = 0.6\%$.

To compute stresses due to a temperature change of 50°F. we have a variation of $.0000055 \times 50 = .000275$ parts per unit length. Then the thermal stress in concrete is $2{,}500{,}000 \times .000275 = 687$ lbs. per sq. in., which is 2 to 3 times the ultimate tensile strength of the concrete. For this variation in length of .000275, the thermal stress in the steel is $30{,}000{,}000 \times .000275 = 8250$ lbs. per sq. in. If we allow the extreme temperature stresses to reach the elastic limit of the steel, then, at the point where the concrete is on the point of producing a temperature crack we have for medium steel with elastic limit of 35,000 lbs. and 300 lbs. of ultimate tensile strength of concrete, the maximum stress that can come upon the steel is its own thermal stress of 8250 lbs. and 300 lbs. of ultimate tensile strength of concrete, i. e.,

$$\frac{35{,}000 - 8250}{300} = \frac{26750}{300} = 89.17 \text{ sq. ins.}$$

of concrete cared for by 1 sq. in. of steel reinforcement or 1.12%. For high carbon steel with 50,000 lbs. elastic limit we have

$$\frac{50{,}000 - 8250}{300} = \frac{41750}{300} = 139.17 \text{ sq. in. or } 0.70\%.$$

The above theory is based on what available experimental data there are and does not claim to be rigid. It is probable that the compressive stresses induced in the steel by the shrinkage of concrete in setting will neutralize the tensile stresses in the steel due to its contraction under the reduction of temperature in setting. If this is so the area of steel required for shrinkage can be neglected.

It is not considered good practice at present to place more than 1¼% of reinforcing steel for all purposes. It will be noted, however, by the foregoing method, in medium steel, a total of 0.81 + 1.12% or 1.93% is required for thermal and shrinkage stresses alone without regard to the tensile stresses due to load.

Assuming the maximum percentage of steel, therefore, this theory will give a required percentage of 1.12% for thermal stresses alone, leaving but 0.13% for tensile stresses due to load, a percentage so low as to cause prohibitory stresses in the steel. It will be seen that ordinary practice of to-day allows hardly any of its steel for thermal or shrinkage stresses, but that the steel is proportioned only to take the load with the hope that a small extra area may take up some of the expansion and contraction stresses. Therefore, since the amount which can go to expansion stresses is far below an amount as calculated by a conservative method, the whole scheme of taking up all expansion in this way seems ill-advised. The problem is thus thrown back on the second method of solution, viz., of dividing the construction into a series of separated units, each of which is free to move at its ends but may be reinforced in its own length by steel to take up local stresses due to expansion and shrinkage.

EXAMPLES OF EXPANSION AND CONTRACTION.

The following notes are a few of a number which have been made to show the effect of expansion and contraction in a few of the large concrete works, some of which were built as monoliths, reinforced for extra stresses, and in others of which expansion joints were placed. Most of the examples cited are in reservoir or dry-dock work with which the author has had more experience.

BRIGHTWOOD RESERVOIR, WASHINGTON, D. C.—This reservoir is $415 \times 300 \times 20$ ft. deep. It is of plain, unreinforced concrete, with a vertical outer face and a sloping inner face of concrete. The vertical outer face has a heavy earth embankment against it. In section and backing of the vertical wall, it is similar to the side walls of a dry-dock. The concrete wall was built in alternate sections not exceeding 50 ft. long. Vertical key ways 6 ins. square, ½ of which was in each section, were built for expansion joints and filled first with asphalt. The concrete proportions were 1:2½:5. In placing the fresh concrete on that which was already set, the surface was carefully washed and rich mortar was placed in order to make a joint. The forms were left on from one to two days to harden; surfaces which were to be exposed to the air or water had a face of 1:2 mortar, 6 ins. thick, placed at the same time the concrete backing was placed. The concrete floor was laid in two layers, each 5 ins. thick, and laid in blocks 15 ft. square. In the joints was placed a 3-ply felt, up to bevels, which topped off each joint. After the shrinkage cracks appeared, these bevels were run full of asphalt and then 2 ins. of 1:2 mortar was troweled over everything.

The experience with this reservoir is interesting since the asphalt in the key ways cracked and leaked in cold weather, and in warm weather ran out beneath the water surface. The asphalt was finally taken out and replaced with carefully selected puddling clay rammed into the key ways. The contraction of the concrete permitted this material also to run out through the expansion joints into the reservoir and caused leaks to appear through the embankment. After this occurred, the puddling clay was taken out and plain loam, with clay, sand, grass roots, dirt and fibrous material, were rammed into the key ways. This material has successfully prevented leaks up to the present time.

NAVY YARD, NEW YORK, N. Y.—Expansion of sidewalk broke the curbing at the northeast portion of steam engineering machine shop; also, at southeast corner of Building No. 24; also on both easterly corners of Building No. 22.

CHICAGO RETAINING WALLS.—In track elevation work the walls had joints from 50 ft. to 60 ft. apart, which opened up quite noticeably in cold weather. In walls of small cross sections, air cracks opened up half way between. These joints were tight in summer.

RESERVOIR AT COLOMBO, CEYLON.—This reservoir was entirely of concrete, unreinforced, 176 × 160 × 40 ft. high. When the water was turned on, a crack appeared, ¾ in. wide at the bottom, and ½ in. wide at the top, causing a considerable leakage. These cracks were cut down and filled in with brick work. Water was again turned on and further cracks appeared. A mixture of asphalt and lime carbonate was placed between the brickwork and the concrete in the side walls. The repairs caused by these temperature cracks kept the reservoir out of service for about a year and a half.

WENTOW CANAL AT MARIENTHAL, GERMANY.—The protection work here consists of a reinforced concrete slab, the reinforcement having a 6-in. mesh. The slab was 8 ins. thick, 12 ft. wide, 450 ft. long, placed on a slope of 1 to 1. While this work was reinforced and the intention was to make it monolithic, it was so badly cracked that portions of it had to be removed, leaving room for expansion.

GALVESTON SEA WALL.—The base runs up to 4 ft. above mean low water. On the top of this base, in each section, are two grooves or recesses 6 ins. deep × 4 ft. wide to give a key or toe-hold for the upper part of the wall. The upper part of this wall is built in alternate sections. This wall is 5 ft. wide at the top, 16 ft. at the base, and 16 ft. high. The walls were built in sections, the length of the section corresponding to what could be finished up as one day's work. Most of these sections ran from 50 to 55 ft. long; some ran as low as 20 ft., but most of them ran above 40 ft. The base during the latter part of the construction of this wall was connected to the upper part of the wall by means of steel rods imbedded in the base and sticking up through, so that concrete could be placed around same. This was in addition to the grooves before mentioned, which ran parallel to the wall. After the forms were removed, intermediate sections were built. Specifications required "the work on each section to be continuous so as to leave no lines of cleavage. After the forms are all removed, the line of vertical contact between the sections on the front and back sides of the walls shall be tool-pointed, as directed," by the engineer for the sake of appearances.

MELAN ARCH, YELLOWSTONE PARK.—This 120-ft. span arch was specially reinforced over the springing line with additional steel rods, to take care of thermal and shrinkage stresses, but still cracks appeared at both sides of the bridge. That the full danger of such cracks was appreciated was shown by the promptness with which waterproofing was applied to the concrete to prevent its distintegration by frost and the rusting of the reinforcement.

ST. AUGUSTINE, FLA.—Some very large hotels were built of monolithic concrete. Of these hotels, three are known to be very badly cracked from top to bottom. Although they do not necessarily endanger the stability of the structure, the unsightly cracks do injure the appearance very much.

U. S. NAVY YARD, NEW YORK, DRY-DOCK NO. 2.—This structure is made of monolithic concrete in two sections. Where the two sections join, the attempt was made to fasten them firmly together. The concrete of the two sections was dove-tailed and a heavy reinforcement of railroad iron was placed at this joint. This work shows cracks where the concrete placed on different days comes together. Where the intervals between the more prominent cracks are large there are to be found numerous small cracks which run in every direction.

PIER NO. 1, FOR COAL HANDLING PLANT AT U. S. NAVY YARD, NEW YORK, N. Y.—This pier was completed in 1902, and is 500 ft. long, 68 ft. wide, founded on piles backfilled with rip-rap. The pier extends into the East River in from 20 ft. to 45 ft. of water, and is exposed to the heavy currents and ice action due to the bend in the river at the navy yard. No cracks had been noticed in this pier until the latter part of 1906, though they undoubtedly existed last winter. The usual disintegration of the concrete by the frost and ice between half tide and low water is just beginning to show. There are 12 cracks on the south side varying from 1-16 to ⅜ in. in width. On the north side there are 10 cracks; a part of one crack is ¾ in. wide. Twelve of these 22 cracks are 1-16 in. wide. Four cracks not counted in the above are rather fine, being 1-32 in. in width.

U. S. NAVY YARD, BOSTON, MASS., DRY-DOCK NO. 2.—This new 750 ft. dock of granite facing and concrete backing, completed about 1904, shows cracks in each case at or very near the timber slides which are sections of weakness. One large crack ½ in. wide is one-third the length of the dock from the entrance. The other cracks are about 1-16 in. in width or less, and extend in an irregularly vertical plane through the granite masonry blocks when there is no vertical masonry joint near the crack in the concrete backing. In the length of the dock there are eight of these cracks on one side and nine on the other side. Hair cracks or surface cracks are not counted. The dock has a gravel foundation.

U. S. NAVY YARD, PORTSMOUTH, N. H., DRY-DOCK NO. 2.—This new 750 ft. dock, finished about 1904, is founded on solid rock. The east side of this dock has 18 ins. or more of concrete lining to the granite facing masonry. The upper portion of the west side of the dock is clear of the rock, but rests upon it. In the fall of 1902 cracks developed in joints and across the stones of the masonry where the east and west walls had been finished to their completed height. In January, 1903, these cracks were ⅛ in. wide; the following summer they had closed up. In February, 1907, in very cold weather, the first observed cracks were 1-16 in. and 3-32 in. in width.

There is some difference in the cracking of the two sides of this dock. In the 18 ins. of concrete back of the coping, cracks are very frequent; usually they occur opposite joints in the coping; in some places they appear at every joint. Five cracks about 1-16 in. wide appear in the masonry on the east side which has the thin concrete lining to the rock, against seven similar cracks on the west side of the dock. There are other fine cracks and possibly larger ones hidden at present by ice and snow.

MASONRY DOCKS WITH MASONRY BACKING, U. S. NAVY YARDS.—Dry Dock No. 1 at the Boston Yard gives trouble on account of the action of frost causing mortar to drop out of almost every joint. Dry Dock No. 1 at Brooklyn has given no trouble from any cause since the entrance was rebuilt in 1886. Dry Dock No. 1 at Norfolk has some trouble with joint cracks. There seems to be no trouble with the cracking of the granite blocks in these old docks on account of temperature changes, and there is no shrinkage of backing as in concrete. These docks are shorter than the new ones, but they are long enough and old enough to produce contraction cracks before this if there were any tendency in that direction.

EXPANSION JOINTS IN DRY DOCKS.

So far as dry dock design is concerned, the above examples show that joints must be provided not only longitudinally of the dock every 50 ft. or less, but also transversely in the dock floor, since there is a plane of weakness in the drainage culvert under the keel blocks. Note the big crack in Dry Dock No. 2 at the head of the dock, the width of which dock is 62 ft. at the coping and 30 ft. on the floor. Dry Dock No. 2 was built in two sections tied together at joints. Dry Dock No. 4 is to be built in three sections with two expansion joints filled with asphalt.

The ideal material for filling expansion joints is one which is not affected by temperature, and yet which will flow to fill up breaches or openings in the filling material, made by the movement of concrete, that is, must be ductile at low temperatures, and not run out of the joint at high temperatures. These filling materials have been used as asphalt paving cements and some are known to the trade as Byerlite, Pittsburg flux, paraffine residuum, Texas residuum (Hydrolene B), Parahumid, etc. Hydrolene "B" has been used in some vertical expansion joints, or more properly, vertical lines of separation, in Dry Dock No. 3 at the Norfolk Navy Yard. This separation joint does not provide against vertical movement of the adjacent sections of the floor of the dry dock.

A form of expansion joint, if properly reinforced, which would provide to some extent against this vertical movement of adjacent sections, is like that shown in the upper left hand corner of Fig. 1 with the key-way left out, a portion of this joint being in a horizontal plane and a portion in the vertical plane. There is considerable doubt, however, that there is any material which really answers the

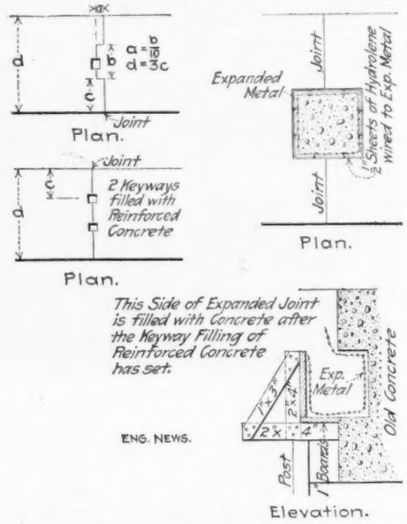


Fig. 1. Some Expansion Joints in Mass Concrete Work.

requirements to expand and yet not flow, for whenever any material is secured which is not affected by changes of temperature it loses its elasticity and ability to flow into adjacent spaces, or as the expert who first suggested hydrolene manufacture puts it, "There is no cement known that shows both ductility and non-susceptibility to changes in temperature." The above-mentioned joint filled with hydrolene of one kind only, would, if the hydrolene is too hard, crack when there is a movement of the concrete, and fail to expand and fill up the open space left on the opposite side of the joint; if it is too soft, the material would slowly run out of the joint, leaving nothing but water or mud to take its place. What is needed is a material which, when it has been compressed during expansion of the concrete, will flow into the space left when the concrete contracts, and still not flow sufficiently to run entirely out of the joint, leaving the expansion joint empty. It is thought that the best way of providing a water-proof filler for expansion joints is to use hydrolenes of different degrees of hardness; that in the vertical plane being in the nature of pitch which can run slowly so as to fill up any cracks, openings, or crevices left in the horizontal plane by the movement of the concrete in the horizontal plane. This might possibly be accomplished by the use of hydrolene "B" in the horizontal joints with a melting point of 200°F., and the use in the vertical joints of hydrolene "B" with a melting point of 172°F., both of these hydrolenes being commercial products.

BLOCK CONSTRUCTION.

There being no ideal material that will compress when concrete expands into the expansion joints, and fill the space left when the concrete contracts from the expansion joint, it leaves open to serious consideration the application to dry dock construction of the method used in dam building. That is, building in alternate blocks varying in dimensions so as to provide: No through joints either vertical or horizontal; no dimension over 30 ft.; no block which cannot be finished by continuous working. This means a mass that retains its position by its own weight. When a 16,000-ton battleship with its concentrated loads is on one side of a joint and the other side has no "live load," the normal upward hydrostatic pressure (dock empty) is increased, especially in soft ground, by the heavy loads on

the concrete and piles of the adjacent section. Therefore the piles should be permitted to stick up from 2 ft. to 4 ft. into the concrete instead of the usual 6 ins., thus developing their "holding down" power as well as their "bearing power." This will result in a slight saving in cost of material.

KEY WAYS.

In case "block construction" cannot be employed, it appears that the best way to provide for expansion is to leave good-sized key ways the entire length of the joint, either horizontal or vertical. Fig. 1 illustrates the ideas mentioned in this paragraph. In the upper left is shown a joint with only one key-way which could be used either in the floor of a dry dock or reservoir, or in the side walls of same. In the case of side walls, the key-way would be vertical and would be filled only after the concrete in the adjacent sections had set, and as late in the season as possible. In the lower left, two key-ways are shown, in place of the offset and one key-way, which is shown in the first joint. The lower is believed to be a simpler arrangement and just as effective against leakage.

A method of constructing these key-ways in a vertical plane is shown in the upper right hand with reinforcement to which is wired the sheet of hydrolene. The reinforcement, such as expanded metal or welded wire could be formed on the ground, wired up, and then lowered into place before pouring. This kind of reinforcement should be as effective as the wire spirals used in concrete column reinforcement, and the angles will prevent water percolation. The lower right hand cut shows a method of making these key-ways in a horizontal plane, such as the floor of a dry dock. The notes on the drawing explain matters sufficiently. Sheets of hydrolene could be used in this case just as well as in the cut above it. In the case of these horizontal key-ways, it would be necessary to build these sections of concrete alternately, building the keys as soon as the concrete has set sufficiently to permit the taking down of the forms. After the keys are all in place, the intervening sections could be placed without using any forms whatever.

PROPOSED SYSTEM OF BLOCK CONSTRUCTION.

It is with much diffidence that any specific ideas on the use of block construction for reservoir walls and dry docks is presented. There are certain conditions in floors of dry docks and reservoirs which are not met with in dam con-

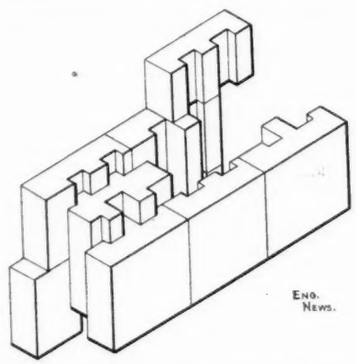


Fig. 2. Isometric View of a Wall Construction in Dry Dock or Reservoir.
(Large blocks and broken joints as a substitute for expansion joints.)

struction. Fig. 2 has an isometric view of blocks which could be used in side walls. Forms would be required for blocks on the front and back of the side walls; the interior blocks could be placed practically without forms except as a cut-off. The interior blocks are stopped at such elevations as to break joints with the blocks on each side of them, and none of the horizontal joints on the two outside sets and the one interior set of blocks are in the same horizontal plane. While this figure presents many angles for the flow of water, excellent bonding, and a perfect system of broken joints, the cost for forms would be considerable, and the use of this idea in a dry dock floor is not at once apparent. Various other forms suggested from the shapes of the ordinary concrete building blocks and adapted to various constructions, will present themselves as occasion arises.

Fig. 4 shows, in plan, two courses of a wall. It will be noted that there are only two shapes used, and that the smaller is the same as the larger, with a cut-off shutter at the third step. The method of procedure would be to lay the first course shown at upper part of Fig. 4, by placing successively the forms and leveling all off at the same height (from 4 to 6 ft.). There will be left on this course an opening between the front and back stones. When this course has set and forms have been removed, the second course (lower part Fig. 4) is placed, in the same manner, on top the first one. Fig. 4 shows the correct longitudinal relation between the two courses and Fig. 3 is made by superimposing one course on the other, thus giving the exact relative position of the stones. In Fig. 3 the first course is shown in dotted lines where invisible. The same two sizes of forms are used in both courses and throughout the work. Wherever the forms in the second course bridge the interior vacant space, concrete will be two courses, or say 10 ft. thick. After the second course blocks are all in, there still remain well holes "G," shown hachured in Fig. 3. These can be filled up without forms, stopping the filling so that the horizontal joints come halfway between the courses, thus making a strong bonding and interlocking.

With a dry dock bottom thick enough to remain in place by its own weight, the method above outlined for side walls could be used by extending same laterally; furthermore, it is believed that bonding in this way would hold a dry dock or lock floor (in, say, three courses) together just as well, or even better, than if there was the usual attempt to make said floor act as a beam, by any method involving the placing of new concrete on the old concrete, as occurs when the concrete placing is done on different days. Forms required in Fig. 3 are very light. Further, the

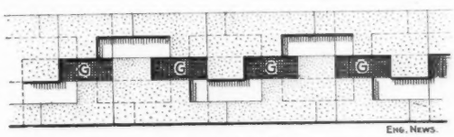


Fig. 3. Plan of a Suggested Wall Construction to Provide for Expansion and Shrinkage.

forms as shown are comparatively small, easily handled, and can be used time after time until they are completely worn out. The bolts for holding the sides of the forms together can be used until worn out, nothing being left in the concrete but the nuts and wire.

One way of avoiding unsightly cracks in bottom and sides of docks, reservoirs and locks is to face the concrete with ashlar masonry. A granite facing would do away with one side of the forms shown in Figs. 3-4, but unless the concrete backing is provided with means of caring for the shrinkage, there may appear a small crack or cracks in the masonry and through the granite itself such as that in the new docks, Portsmouth, N. H., and Boston, Mass. The old stone docks built up entirely of masonry, in Boston, New York, League Island and Norfolk do not show these cracks, but they are comparatively short. The masonry aqueducts of the Erie Canal, one at least over 1,000 ft. long, do not show these cracks in the masonry where it is thoroughly bonded, although the coping shows a "creeping" movement of several inches per year.

From the examples given, it is believed that a fair case has been made out in favor of alternate blocks or block and bonded construction of concrete backing over the present lack of method in preventing cracks in facework of docks, locks and reservoirs. It seems proper to assume that when reduction of temperature causes strain in masonry that these cracks are to a great extent concentrated in the mortar joints and beds in the masonry and in the constructive joints between blocks in concrete backing. When the mass is entirely of well bonded masonry, cracks are distributed uniformly in joints so as not to be unsightly, and with well bonded concrete blocks the same effect may be expected if the size of individual blocks is not chosen too large. In case of masonry dock lining in which joints like those shown in Fig. 1 are used, it would be necessary to omit mortar from the joints of individual stones which cross the expansion joint, leaving a toothing such as is done when new brick work is joined to old brick work. The

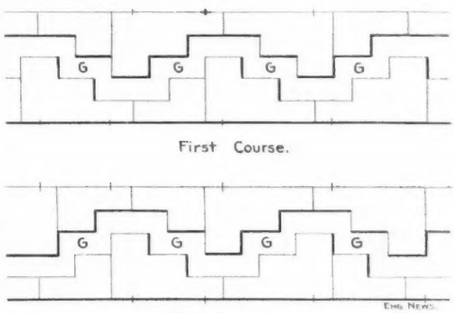


Fig. 4. Courses in Detail which go to Make Up Wall Construction Shown in Fig. 6.

mortar joints can be pointed up after a few years or when desired to avoid frost troubles. Similar attention should be paid to timber slides, and to the arrangement of vertical constructive cracks in the alternate block construction of Figs. 3-4 Cyclopean rubble for the block construction should produce favorable results. On the other hand, the method of expansion joints shown in Fig. 1 appears less complicated, and more scientific at first glance, if the partially open joint in masonry, as mentioned above, is used at the expansion joint.

TESTS OF FIRE-KILLED TIMBER have recently been made at the timber-testing laboratory of the U. S. Forest Service at the University of Washington (Seattle). Green and fire-killed white fir (Abies concolor) from New Mexico were tested. According to a circular issued on April 1 by the U. S. Forest Service, the results indicated that the killing of the timber by fire is not in itself injurious, and that sound fire-killed timber should simply be considered thoroughly seasoned timber, so far as its use is concerned. Fire-killed timber checks badly, however, when left in the tree, and this is an obstacle in the way of utilizing it for some purposes. For the manufacture of lumber it should be used, if possible, within a year after it is killed, but satisfactory railroad ties have been made from timber killed fifty years previously.

THE CEMENT-TESTING LABORATORY OF THE U. S. Reclamation Service has been moved from Denver, Colo., to Chicago, Ill., where it will be installed in quarters provided through the courtesy of the Armour Institute of Technology. The work carried on by the laboratory consists principally of acceptance tests of the cement purchased for use on the construction work of various projects of the Reclamation Service and of the supervision of the shipment of this cement from the factories after the tests for acceptance have been made. The field covered by this work includes all of the projects of the Service, with the exception of those situated in the Pacific Coast states, for which the supply of cement is obtained from plants in California and tests of which are made at the laboratory of the Engineer of Soils at Berkeley, California, and also of the Salt River project in Arizona, at which point the Service has a cement plant of its own for making the cement required on the project. In addition to the regular tests for the acceptance of material which constitutes the routine work of the laboratory, some experimental work has been carried on, such as tests of sand-cement and tests of sand and stone for concrete materials on various projects. The laboratory is also equipped with an outfit for making tests of the permeability of cement mortar under water pressure. It may be noted also that, in addition to the regular tests for tensile strength made in the laboratory, occasional sets of long time tests are made on each brand of cement tested, these sets being for various periods running up to 10 years, and there are now over 3,000 briquettes in storage for these long time tests.

REINFORCED-CONCRETE TOWERS FOR HIGH-POTENTIAL TRANSMISSION LINE.

By F. W. SCHEIDENHELM.*

At Brownsville, Pa., there has recently been completed a novelty in engineering construction in the form of two reinforced-concrete towers. This construction, in view of its apparent success, opens a new field in reinforced-concrete work, especially along the line of its application in this particular case. So far as the writer has been able to learn, these are the first reinforced-concrete towers to have been built in place. At St. Catharines, Ont., early in 1906, Mr. J. L. Weller built a 150-ft. tower for a wire crossing over the old Welland Canal. This, however, might better be termed a reinforced-concrete pole, for its dimensions are such that it was possible to build it horizontally on the ground and then hoist it into its upright position by means of a pair of shear legs.†

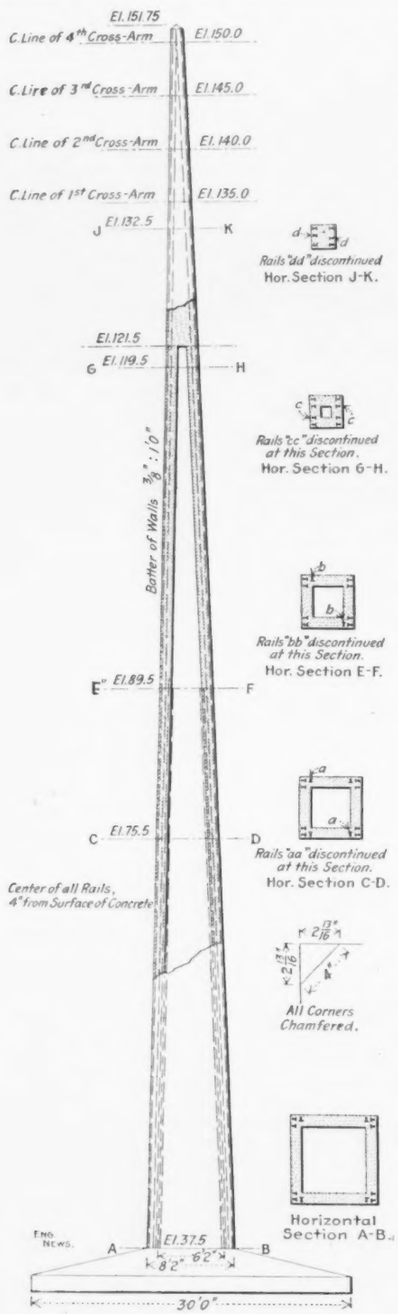


Fig. 1. Elevation and Section of Reinforced Concrete Tower for High Potential Transmission Line.

The main tower of the project in hand rises 115 ft. above its foundations and supports one end of the West Penn Railways Co.'s transmission line crossing over the Monongahela River at Brownsville, Pa. A second tower, but 55 ft. high, acts as an anchorage in taking the direct strain of the main span.

PROBLEM.—The problem solved by the construction of these towers was that of supporting a 1,014-ft. span across the Monongahela River at such a height as to allow the prescribed government clearance for navigation. Since the river is subject to heavy floods, the minimum height of cable above low water was fixed at 79.5 ft. On the Brownsville side of the river the span anchorage is in the local sub-station, a strongly-built structure of brick, concrete and steel. On the West Brownsville side, however, some form of tower was found necessary.

The first question was to decide between reinforced concrete and steel. After an extended discussion, involving comparative estimates of cost and considerations of maintenance, reinforced concrete was given the preference.

The second question was as to the advisability of building one tower or two towers. In case but one tower had been built this tower would have taken not only the wind strain on the main span, but also the tension of the cables, and that at an average height of 105 ft. above its base. Adequate guys were impracticable, hence in either case the structures had to be self-supporting. However, on the basis of several comparative designs, the following construction was decided upon as being the more economical:

DESIGN.—The main or higher tower was built as close to the river bank as practicable and designed to take as a maximum load only the wind stress on the tower itself and the weight of the wires. The shorter tower was then built about 230 ft. back of the main tower, and the cables were, as has been stated above, brought down to anchor upon it.

In order that the main tower might be designed for practically the wind stress alone, a special roller-bearing saddle was designed for carrying the cables over the tower without rigid connection to it. In addition to the wind stress, the main tower receives minor stresses in the direction of the span and a direct vertical compression. The stresses in the direction of the span are (1) a component of the stress in the cable, due to friction in the roller bearings; (2) a horizontal component of the resultant of the stresses in the cable, this resultant being due to the fact that the cable from the main tower to the anchorage has a much lower dip than that from the main tower to the station across the river. The direct compressive stress on the main tower is composed of the vertical component of this same resultant (which is a maximum when the wind stress is a maximum), and of the dead load of cable weights and ice on the cables.

The cables were assumed to be liable to the extreme condition of an ice coating equal in thickness to the diameter of the cable, with a wind stress of 20 lbs. per sq. ft. upon the resulting projected surface (equal to three times the cable diameter). The wind load on the main tower used in connection with the resulting stress due to wind on the cables was assumed to be 40 lbs. per sq. ft.

In computing the stress in the cables themselves they were treated as catenaries. The maximum unit load was therefore the resultant of the weight of the cable and the ice in a vertical direction and the wind load in a horizontal direction. The maximum allowable sag was 36.6 ft., while the assumed temperature range of 115° brought the minimum sag down to 33.4 ft. Hence the anchorage tower was designed to carry a pull of 122,000 lbs. at an average distance of 38.5 ft. above the base. In this case the stress due to the wind, acting transversely to the cables on the anchorage tower itself, was found to be amply taken care of after the tower had been designed to take the direct pull of the cables.

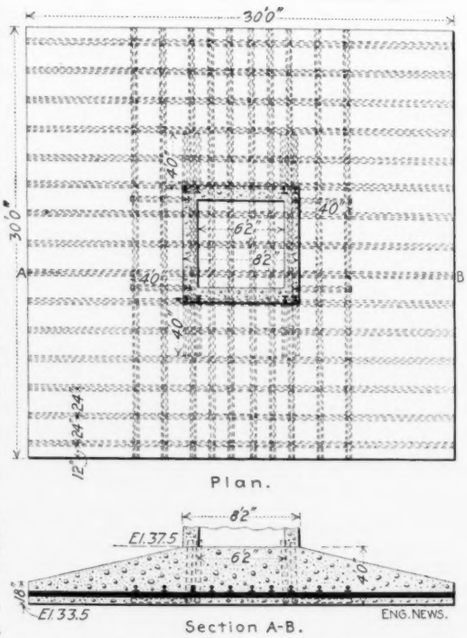


Fig. 2. Foundation of Reinforced Concrete Tower.

Both towers were designed as cantilevered beams. The main tower, however, is liable to maximum stresses in directly opposite directions, depending upon whether the wind is blowing up stream or down stream. This fact made it necessary to insert additional steel, and thus reduce the economy of a reinforced concrete section, which should ordinarily receive stress in one direction only. To offset this reduction in economy it was determined to make the section a hollow square with walls 1 ft. thick. Though the section is square, yet owing to the position of the reinforcement it is somewhat stronger along the axis parallel to the wind stress than in the transverse direction.

At its base the main tower is 8 ft. 2 ins. square. A uniform batter on all sides makes the tower pyramidal in form up to its top, where the section is practically 1 ft. square. The walls, as stated above, are 1 ft. thick up to a point 84 ft. above the base, where the section becomes solid.

The anchorage tower, on the other hand, is a reinforced-concrete structure in the strictest sense. Each horizontal section was treated as a beam to withstand a bending moment, which does not reverse in direction. Hence the fullest economy was obtained and a solid section was used, since no surplus steel was required in order to provide for varying conditions of stress. This tower is 4 ft. by 10 ft. at the base, and batters up to a section 1 ft. square at 41 ft. 1 in. above the base, the tension side being vertical. Above this point the tower is of uniform section up to its full height of 55 ft.

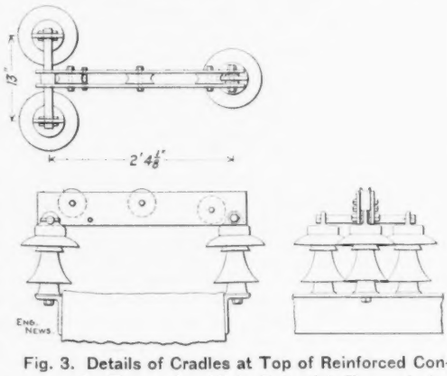


Fig. 3. Details of Cradles at Top of Reinforced Concrete Tower for High Potential Transmission Line.

*Connellsville, Pa.

[†In Engineering News for Jan. 4, 1906, there is a lengthy article by Mr. Joseph Mayer on "The Structural Design of Towers for Electrical Power Transmission Lines" which is a concise and complete treatment of the theoretical side of the construction of such towers as are described in this article. It has also been issued in separate form as an Engineering News reprint.—Ed.]

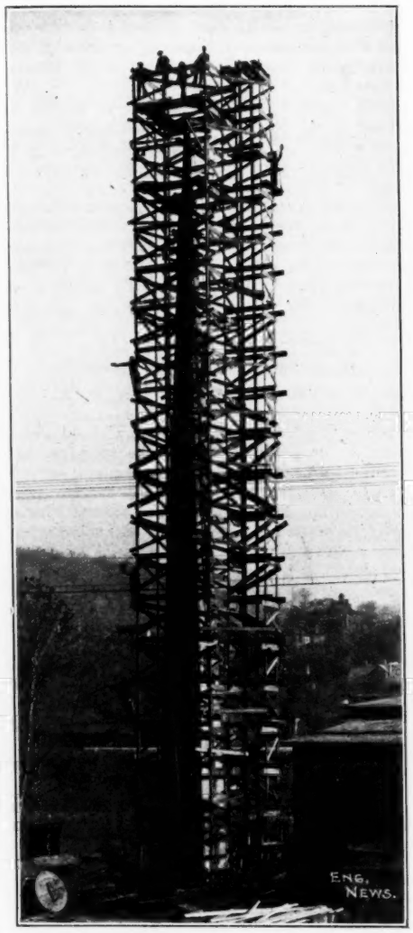


Fig. 4. Falsework on Concrete Tower. Falsework 120-ft. Tower Completed to 102 ft.

REINFORCEMENT.—For reinforcement old T-rails were used. All of the rails forming the vertical reinforcement were of 60 lbs. section. The safe unit stresses were cut down to allow for the wear which many of the rails showed. Incidentally the use of rails solved the problem of end-to-end connection in the case of the vertical reinforcement, for the ordinary splice-plate joint thus became possible. On the other hand, the large cross-section of each rail was a disadvantage. In certain sections of the towers, for instance, it was necessary, under the circumstances, to insert the full cross-section of a rail, even though only a fraction of it was required by the stress to be carried. The base section of the main tower contains twelve 60-lb. rails, three being placed at each corner, while the base section of the anchorage tower contains ten rails in the tension side and two in compression. Thus the main tower base contains 1.73% of steel, and the anchorage tower base 1.25%.

In addition to the vertical reinforcement of rails, a spiral winding was used. Here again economy was practiced by the use of discarded ⅝-in. cable, which was obtained at various ferries along the river. Two spirals were wound, 1 ft. apart, thus giving a 2-ft. pitch. Tie wires secured the spiral winding to the vertical reinforcement.

FOOTINGS.—Test pits showed that there was no rock near enough to the surface in which to anchor the towers economically. Hence this original intention had to be abandoned and spread concrete footings for both towers were substituted. For the main tower sub-soil the safe loading was taken at 1 ton per sq. ft., and accordingly a base 30 ft. by 30 ft. was required to prevent overturning. Because of comparatively weak soil found under the main tower footing, openings were left to allow for driving piles through the footing in case this should be found necessary in the future. Under the anchorage tower, however, a firm clay was found. The safe load for this clay at the toe of the footing under maximum stress was taken as 2½ tons per sq. ft. This allowed the use of a footing 10 ft. by 31 ft. Both footings taper to the outer edge, where the thickness is 18 ins. and 12 ins., respectively.

The footings were reinforced by rails, which were placed 2 ft. c. to c. (Fig. 8). Two tiers of rails in the form of a grillage were used to properly distribute the stress. All tower rails which enter the footings and which are liable to take tension were bent nearly at right angles to form a 4-ft. hook. This hook was then inserted beneath the lower tier of footing rails. Thus an admirable bond was formed between tower and footing in each case.

Rough forms were used in putting in the main tower footing. In the case of anchorage footing, however, the walls of clay, after excavation, allowed the concrete to be put in without any forms.

MATERIALS. — Gravel concrete, 1:2½:5 mixture in footings and 1:2:½:4 mixture in walls, was used and delivered to the work by barges and a temporary tramway. The concrete was put into the tower forms so wet that it would readily flow, and the result was a finish which required practically no attention to give a smooth and pleasing appearance, being almost white after thoroughly drying.

MACHINERY.—The entire work was done by the West Penn Railways Co. itself, and hence the use of motors was found to be extremely economical. Current was obtained from the local secondary system. Two Wagner single-phase 220-volt motors were used. One, of 7½ HP., drove, by belt, a Williams friction hoist. The other, of 10 HP., drove a No. 1 Ransome mixer, also belt connected. The hoist was used alternately in connection with the tramway and for running an elevator. The elevator was large enough to take one large wheelbarrow and a man comfortably. It was intended only for hoisting concrete, but it proved to be safe enough to take three men at a time, and so was actually a great time-saver from this point of view alone. A ⅝-in. steel cable passed from the elevator directly over the crown-block pulleys at the top and down to the hoist. The elevator shaft was attached to the main falsework, and was built up along with it.

FALSEWORK.—The falsework in both cases was 12 ft. square, and consisted of four corner posts of yellow pine, 2 × 4 ins., lapped and butted, with 1 × 12-in. yellow pine plank every 6 ft. for horizontal chords and 1 × 6-in. hemlock X-bracing. Additional minor bracing was provided to support the working platforms and elevator.

The falsework was kept sufficiently in advance of the forms to allow of the hoisting into place of the 30-ft. rails. These were brought in through the falsework by means of block and tackle attached to a platform at the top. As the work progressed the falsework was securely braced against the completed portions of the tower. Thus the falsework was never unsupported for a greater height than 50 ft. At no time was a guy required, even though the wind was several times so strong as to make it unsafe for men to work aloft.

For the anchorage tower the falsework was carried at once to its final height of 60 ft. Because of its lesser height, the work on this tower was necessarily much simpler.

FORMS.—The construction of the forms is best illustrated by the view shown in Figs. 5-6.

Three completed forms, such as shown, each 6 ft. high, were built for each tower, there being sufficient good material left over from the main tower forms to make up the anchorage forms. One form was filled each day; more was impossible, except very near the top, because of the great pressure created at the bottom of each form by a 6-ft. head of wet concrete. Accordingly each form was removed on the third day after it was filled. This left the freshly exposed concrete rather "green," but great care was exercised not to disturb it, and no ill-effects followed.

FIG. 5. EXTERIOR FORM FOR MAIN TOWER.

After stripping, each form had to be cut down to the proper section and reassembled. This was most easily and quickly accomplished by lowering the form to the ground, where the carpenters had ample room to work, and then hoisting it into its new position. The cutting down of the form to its new section was a very simple matter, for the relation between width of lagging, height of form and batter of tower was such that it was only necessary to remove three pieces of lagging from both ends of each side of the form. The wales were then sawed to a shorter length, new bolt holes were bored, and the form was ready. Each form was given a thin coating of motor grease before filling.

The steps for the main tower are of ¾-in. irons, 1 ft. c. to c., with a 14-in. tread and a drop of 1½ ins. to prevent slipping of the foot.

The forms and the reinforcing rails were brought into line separately by means of a tran-

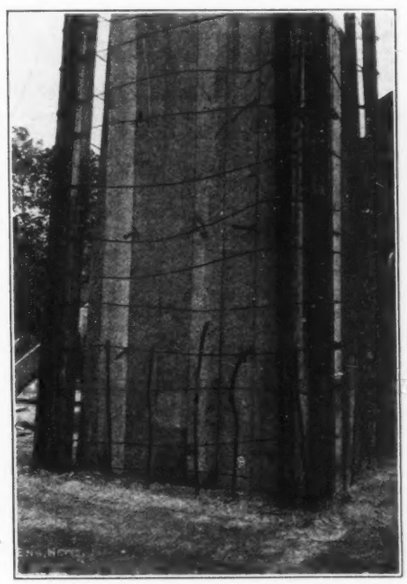


Fig. 6. Interior Form for Main Tower, Showing Position of Rails and Spiral Reinforcement.

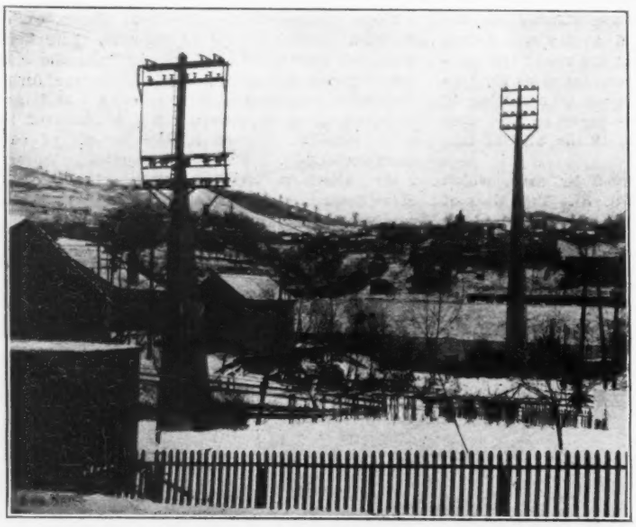


FIG. 7. FINISHED REINFORCED CONCRETE TOWERS FOR HIGH POTENTIAL TRANSMISSION LINE.

sit point on each side of the tower. The rails were temporarily fastened to a wooden frame of the proper dimensions. This frame was centered and then held in place by means of guys. The forms in turn were shifted and centered by wedging against the rails. The rigidity obtained in this way was such that the centers of the forms were rarely out by ¼-in. after filling.

The interior forms, used for the main tower only, were very simple, being composed of hemlock sheathing, backed up by 2 × 4-in. bracing. They were left in the structure.

ARRANGEMENT OF WORK.—The Ransome mixer was set up just at the side of the main tower footing. In concreting this footing a Ransome cart was used to deliver the concrete from the mixer to the different parts of the footing. For carrying the concrete from the mixer to the anchorage footing, about 250 ft., one-horse carts were used, and the concrete was dumped directly into the excavation.

The filling of the tower forms, of course, occupied relatively little time compared with the work of placing reinforcement and forms. The concrete was dumped, or rather poured, directly from the barrows into the forms from a platform, which was on a level with the top of the forms. While filling the anchorage tower forms one-horse carts were again used, the concrete being reloaded into barrows at the foot of the elevator shaft.

Additional bond between courses was provided, besides vertical reinforcement and spiral winding. This consisted of 3-ft. sections of 1-in. cables, which were inserted for half their length at 6-in. intervals in the top of each section after the form was filled. With the added precaution of roughening and wetting the top of each course before the succeeding form was filled, a bond was obtained such that the actual joint can hardly be detected.

The time from the filling of the first main tower form to the completion of the tower was 25 days, while the corresponding time for the anchorage tower was 8 days. Owing to the nature of the work, only a limited number of men could be worked to advantage. The average number of men employed was about 15.

ELECTRICAL EQUIPMENT. — The towers were designed to carry 16 cables, of which 14 have been put up and are in service. All are of silicon bronze, and were furnished by the John Roebling Sons Co. Four carry the 22,000-volt three-phase circuit (one being a spare cable), and are made of 61 No. 15 wires. The other ten cables carry 6,600 and 2,200-volt circuits, and are made up of 19 No. 15 wires. The cables are approximately ½-in. and ¼-in. in diameter.

As has been stated above, at the main tower the cables are supported by roller-bearing saddles. Each saddle consists of a number of old trolley wheels, which were refilled with a soft metal to provide a bearing which would not wear the cables (Fig. 3). They are set between plates, which are fastened by means of bars to four cast-iron caps. These caps in turn are cemented on top of insulators. The latter are of the special design of the West Penn Railways Co., manufactured by the C. S. Knowles Insulator Co. They are mounted on Locke cone-pins.

The main tower has cross-arms made of 4 × 6 × ½-in. angles, braced vertically at the ends of the arms by 3½ × 5 × ⅜-in. angles.

At the anchorage tower the ½-in. cables are "dead ended" in a rope socket joint in an iron yoke. This yoke is connected by loops of ¼-in. silicon bronze cable to two Locke No. 606 strain insulators. A balance wheel guarantees equal distribution of strain upon the insulators. The insulators are set in tandem, and are supported by ¾-in. bolts, with pipe and fiber bushings, between cross-arms consisting of 15-in. 33-lb. channels. The channels in turn transfer the stress to the tower by means of 1¼-in. bolts. Vertical rigidity is furnished by angle bracing, while horizontal adjustment is permitted by eye-bars with turnbuckles. The ten ¼-in. cables are tied directly to Knowles Type T strain insulators, which are held between cross-arms consisting of 12-in. 25-lb. channels. These transfer the stress to the tower proper in the same manner as do the larger channels. The insulation is protected from lightning by horn arrestors on top of the anchorage tower.

Wires were strung early in January of the present year, and the towers have been in uninterrupted service since Jan. 20. During the month of December river floods twice reached a height of 6 ft. above the base of the main tower, and a January flood reached a maximum elevation of 18 ft. above the same base. None of these floods affected the towers or footings in any manner that could be detected by eye or instrument. Since the wires were strung the towers have also been subject to severe wind load without apparent damage. In view of these tests, therefore, the construction is an admitted success.

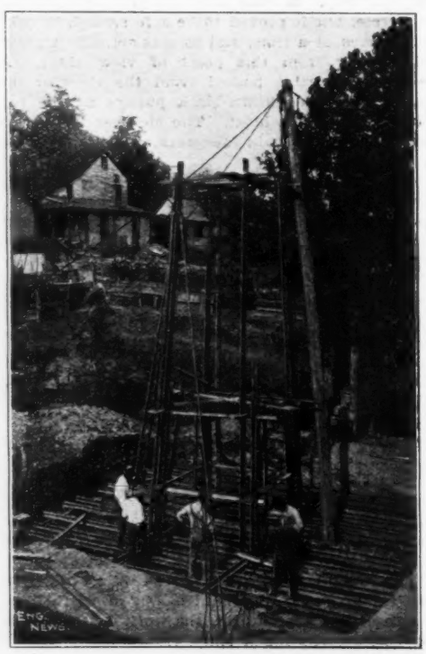


Fig. 8. Footing for Reinforced Concrete Tower.

That the structures are elastic is readily apparent to one on the towers during even a moderate wind. The vibratory movements which are felt, however, can hardly be distinguished by means of a transit.

The work was done under the general direction of Mr. J. S. Jenks, Superintendent of Transmission for the West Penn Railways Co. The tower superstructures were designed by Mr. L. O. Veser, while the towers themselves were designed and constructed by the writer, both of the same company.

AN AUTOMATIC SOUNDING RECORDER.

Mr. E. S. Wheeler, M. Am. Soc. C. E., Chief Assistant United States Engineer, has recently perfected a self-recording sounding device which should be of great service in the sounding of a river or harbor where a great amount of area is to be covered. The following description of the device is taken from a paper by Mr. Wheeler in No. 23 of Occasional Papers, Engineer School, U. S. Army.

The apparatus is called a "Bathometer." In its construction, use has been made of the following principle: If air be forced through a tube the lower end of which is submerged in water, then the pressure of the air at the upper end will equal the pressure of the water at the lower end plus the pressure due to the frictional resistance of the air passing through the tube. If then the pressure of the air at the upper end of the tube and the frictional resistance are known, the pressure of the water at the lower end becomes known, and since the depth of water is a known function of the pressure, the depth is known.

The essential parts of this bathometer are: (1) A flexible tube weighted at one end so that it can be submerged, and branched at the other end so that an air pump and a pressure gage or register can be attached. (2) An air tank and pump capable of forcing a continuous stream of air through the tube and out at its lower or submerged end. (3) A pressure gage that will measure separately the pressure due to the friction pressure at the lower or submerged end. (4) A register that will record automatically and continuously the pressure at the upper end of the tube (expressed in feet) due to the depth of the submerged end. Fig. 1 shows the apparatus with the parts assembled as they were used on the U. S. S. "Hancock." The tube or sounding line is shown at G. It is made of a tube ½-in. in diameter with a bore of ¼-in. The tube is protected by a woven cotton covering, the warp of which is longitudinal or parallel to the rubber tube; this prevents excessive stretching and collapsing that might be the case if the warp was woven in a spiral around the rubber tube. The line used was four-ply, or had four concentric coverings of cotton dipped in a preservative mixture, which gave the line a tensile strength of a little more than 1,000 lbs., and an exterior diameter of 0.85-in. Outside of the cotton there is a metal armor which serves the double purpose of weighting the submerged end and protecting the line from wear. The metal armor is made of short pieces of pipe about 1½ ins. long and ¼-in. thick, which makes the outside diameter 1.35 ins. These short sections of pipe when made of iron are cast and chilled. Between the sections are rubber rings which serve the double purpose of keeping the line more flexible and preventing the armor from slipping along the line. At the lower or submerged end of the line there is a check-valve which permits the unobstructed outward flow of the air and prevents the water from flowing back into the tube. The line used last season was 190 ft. long and about 50 ft. of the submerged end was armored with lead; this made a weight of about 130 lbs. The air is first forced into the air tank D, Fig. 1. When steam was available, a small Westinghouse air compressor C was used. At other times a hand pump was employed. The air passes from the tank into the register A and A' and through the sounding line G. The register is automatic and is shown in detail on Fig. 2. There is a paper disk or a card I which is revolved by clockwork K. The index or pointer carries a fountain pen L, which leaves a continuous line or curve on the disk as it revolves under the pen. The pen is moved by the Bourdon tubes M. Several Bourdon tubes or "tube springs" are attached to one shaft in order to give enough force to move the pen over the surface of the paper disk. The register used last season had eight tube springs.

The adjustments of the apparatus are made as follows: A suitable scale is first marked with circles on the paper disk I. The levers N are then adjusted so that the pen will move a unit on the scale when the submerged end of

the sounding line is changed a unit. This adjustment is tested by submerging the sounding line to a known depth and noting that the pen marks a corresponding depth on the scale. This adjustment when properly made is quite permanent. It is, however, not alone necessary that the pen should move over the proper distance, but it should point to the proper graduations on the scale. The second adjustment is to insure this result and may be called the adjustment for index error. When the open end of the sounding line is out of the water and the pen points to the zero of the scale, there is no index error. The three principal sources of index error are change of pressure in the air tank, dimensions of the sounding line, and area of the aperture. The apparatus has therefore been so arranged that while it is in use these three conditions can be kept so nearly constant that there is no appreciable index error. When not in use, the pen usually drops a little below the zero of the scale. The friction of the air in passing through the sounding line when its open end is out of water causes some pressure in the register. If air is admitted to the sounding line and register in exactly the right quantity, this frictional pressure will cause the pen to rise to the zero of the scale. If, now, the orifice between the tank and the register, the pressure in the tank, and the dimensions of the sounding line remain constant, the pen will remain at the zero of the scale, and the apparatus will be in adjustment. These three conditions are kept constant in the following manner: (1) The pressure in the air tank is controlled by a governor. (2) The orifice between the tank and the register is a small circular hole in a diaphragm which is inserted in the connecting tube. This hole is sometimes jeweled to prevent rust or wear. The size is determined by repeated trials. (3) The dimensions of the sounding line (length and diameter of the bore) are constant unless the line is changed, in which case a new adjustment for index error is made. With ordinary care the index error need not exceed two-tenths of a foot.

The methods employed last season in the use of the "bathometer" were as follows: The work was done from the U. S. S. "Hancock." The depth of water was usually not more than 30 ft. Air was pumped into the tank so as to show a pressure of 70 lbs. All adjustments were carefully made. The sounding line was passed overboard and about 180 ft. paid out. The steamer, running at the rate of about 6 miles per hour, dragged the line over the area to be sounded. The register showed a correct cross-section of the bottom, as at P, Fig. 2. The lines of soundings in Lake St. Clair work were located by means of buoys which were placed in two lines about 3,500 ft. apart on either side of the proposed channel; the buoys were located with transit pointings and sextant angles. In the Lake Huron work the steamer was located every five minutes by transit pointings from two shore stations.

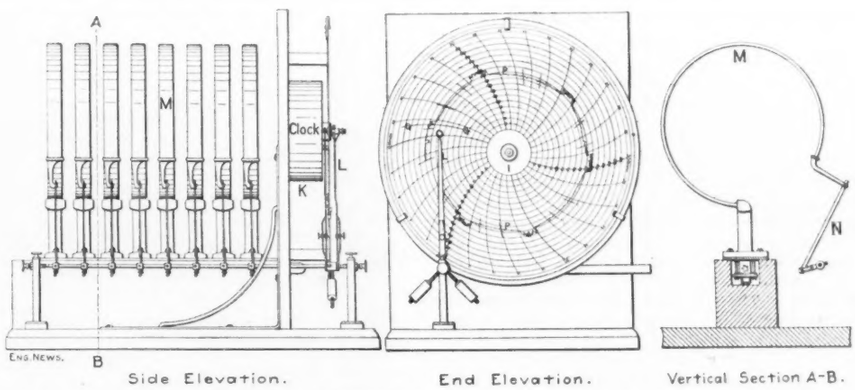
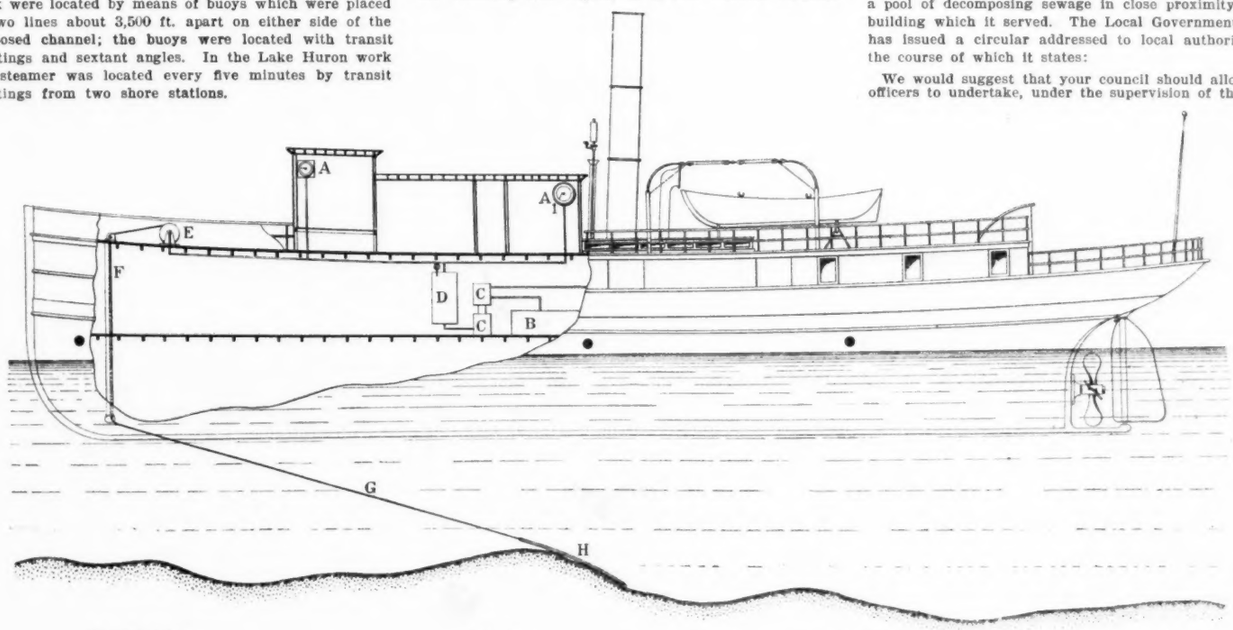


FIG. 2. GENERAL DESIGN OF RECORDING DEVICE OF BATHOMETER OR AUTOMATIC SOUNDING RECORDER.

When more minute surveys are required, such as making estimates for contract work, etc., more accurate location can be made by any of the usual methods. A specimen of the automatic record is shown at P, Fig. 2. This record was made on Aug. 16, on which date the steamer "Hancock" was sunk by collision in Lake St. Clair. The drawing is an exact copy of the disk that was in the bathometer at the time of the sinking. The long straight line at Q was caused by the shock of the collision. The clock was adjusted so as to cause the disk to make a complete revolution in one hour. During the hour preceding the sinking of the steamer, the channel had been crossed from buoy to buoy five times. The time of passing the buoys is shown by the short straight lines, which are made by the observer giving the bathometer a shake or jar so as to cause the pen to vibrate. During a portion of last season 22 miles of ship channel in Lakes Huron and St. Clair were sounded with a bathometer. The work was done rapidly and with a small working force. The bathometer was used in weather that was much too rough for the ordinary methods. The results were as accurate and satisfactory as those obtained in the ordinary methods.

AN EXHAUST STEAM PLANT for the utilization of exhaust steam by the Rateau patent process has been installed at the steel works of the International Harvester Co., South Chicago, Ill. The exhaust steam from a reversible blooming mill is utilized in a 600-KW. low-pressure Rateau turbine, direct connected to two 250-volt direct current generators. The turbine operates at atmospheric pressure (absolute), with a vacuum of 28 ins. The exhaust of the blooming mill is discharged into a water type Rateau steam regenerator, which has a capacity sufficient to take care of intermissions in the operation of the blooming mill engine, of several minutes duration. After passing through the turbine the steam is condensed in a barometric condenser. In order to avoid back pressure on the engine a relief valve discharges the exhaust steam to atmosphere, whenever the pressure raises. During periods when the blooming mill is shut down the turbine is operated with live steam from the boilers, the pressure being lowered by passing it through a reducing valve. This installation uses only about half of the exhaust steam of the blooming mill. It was contracted for and put in operation by the Rateau Steam Regenerator Co., 215 Wabash Ave., Chicago, Ill., who have exclusive rights in America for the Rateau process. The plant was designed by Mr. A. Leonhauser, Engineer of the International Harvester Co., and Mr. C. H. Smoot, of the Rateau Company. It is said to be the first installation in America, using the Rateau process.



A, A, Registers
B Boiler
C Steam-chest and Air-compressor
D Tank
E Cable Drum
F Pipe
G Sounding-line
H Sounding-line weighted with metal armor.

FIG. 1. AN AUTOMATIC SOUNDING RECORDER IN USE ON THE GREAT LAKES. VIEW OF STEAMER EQUIPPED WITH DEVICE.

THE INTERCEPTING OR DISCONNECTING TRAP, long officially held as an absolute necessity to health protection in England, is to be investigated by the Local Government Board. This trap differs from the main or house drain trap, sometimes but not always used in this country, in that there must be provided in England, in addition to a water-sealed trap, a chamber connected with the outer air, so designed as to provide an absolute break in the continuity of the passage between the street sewer and the house plumbing system by means of the sewer connection and house drain. The English intercepting trap was officially adopted some 30 years ago as a result of the widespread public agitation that followed the death of a member of the royal family from scarlet fever or diphtheria, the infection having been attributed to so-called sewer gas. For a number of years past there has been considerable opposition to the intercepting trap, it being alleged that it did more harm than good, giving rise to stoppages and in any case providing a pool of decomposing sewage in close proximity to the building which it served. The Local Government Board has issued a circular addressed to local authorities, in the course of which it states:

We would suggest that your council should allow their officers to undertake, under the supervision of the medical officer of health, the systematic inspection of as large a number as may be practicable of easily accessible inspection chambers at intercepting traps, in order to ascertain facts as to their condition; and that these facts should be recorded so as to show, primarily: (1) The number of traps examined, (2) the number of cases in which (a) the intercepting trap is blocked, (b) an accumulation of sewage is in the chamber, (c) though no accumulation at present exists, appearance of the chamber shows that there has been accumulation, (d) the "cap" of the raking or cleansing arm is displaced. It might be useful, also, if in a few selected cases of obstruction of the trap steps were taken to ascertain the exact cause of the obstruction.

SOME THREE-HINGED CONCRETE ARCHES IN GERMANY.

The use of metal hinges in masonry arches is very common practice in Europe, though but very few such bridges have been built in this country. The three bridges of this type herein described, which have been finished during the past year, are located in Bavaria and carry railways across the river Iller, a tributary of the Danube. The first is at Lautrach and the other two replace an iron bridge which is now off the right-of-way owing to the construction of a new railway station at Kempten. Each is built of concrete with cast-steel hinges at crown and haunches. The cost of the Lautrach bridge was $21,600 and of the two at Kempten $275,100, the corresponding estimates for steel bridges at the same places being $26,200 and $338,000, respectively. The loads under which they were designed are:

	Locomotive, lbs. per lin. ft.	Cars, lbs. per lin. ft.	Concrete: Mass, lbs. per cu. ft.	Concrete: Filling, lbs. per cu. ft.
Lautrach	2,950	2,400	160	107
Kempten	5,480	2,560	160	107

It is interesting to note that "Le Genie Civil," from which most of this information is taken, in commenting on the bridges, says that on account of economy the decorative features of the bridges were kept very plain. To the American engineer it would seem that this lack of superfluous ornament is an advantage to be desired rather than an evil forced by scarcity of funds, and that these arches present a much more pleasing appearance than the common extra decorated masonry arches of continental Europe.

LAUTRACH BRIDGE.

The Lautrach Bridge (Figs. 1-2) consists of one main arch of 187.5 ft. span between hinges and 32.2 ft. rise, approached on each side by one small arch. It is 13.8 ft. wide and is on a grade of 2¼%. The main arch carries the roadway by five symmetrically arranged spandrel arches tied back, as shown, into the main piers by old rails. The hinges are made of cast steel and are of the same general type as the ones described later on the other bridges. The hinges are not masked

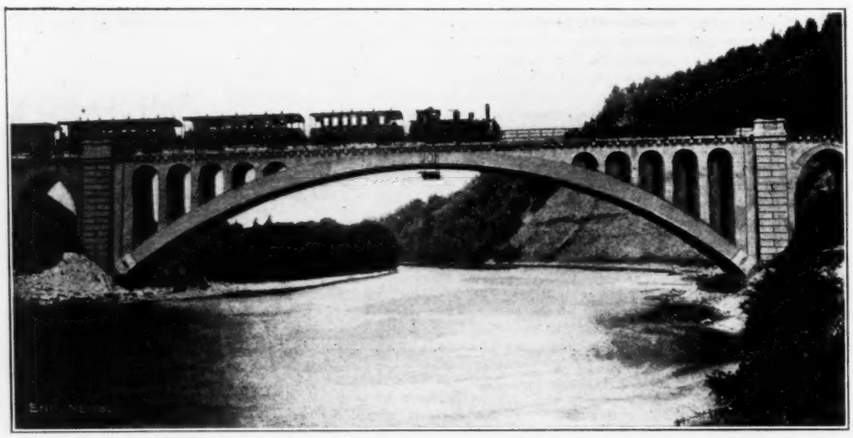


FIG. 1. A THREE-HINGED CONCRETE ARCH ACROSS THE RIVER ILLER AT LAUTRACH, BAVARIA. SPAN, 187.5 FT.; RISE, 32.2 FT.

86.7 ft. The haunch hinges are placed higher up the arch ring, making a span of 166.0 ft. and a rise of 29.0 ft. The main arch rings have a thickness of 4.4 ft. at crown, 6.6 ft. at haunch hinges and 6.1 ft. at spring. The smaller arches have a span of 70.8 ft. and are also constructed with three hinges.

As shown in Fig. 4 the face walls are carried up plain, but the masonry is lightened by the use of hidden spandrel arches, filled over with a lean slag concrete, 1:12 mixture, which transmits the pressures from the roadway to the arch wing in a more determinate manner than earth filling.

The hinges in the main arches are placed as shown in Fig. 4 and extend for the entire width of the bridge. At the crown hinge (Fig. 7) a cover plate of iron is placed over the joint as a protection against water leakage. This plate is itself covered with asphalt upon which rests the ballast for the track. The hinges for the small arches are made of plates of lead 5/16-in. thick laid on blocks of concrete (Fig. 8). Tests on this lead show a compressive strength of 6,044 lbs. per sq. in., and the computed stress on the hinges is 410 lbs. per sq. in. at crown and 680 lbs. per sq. in. at spring.

	Material.	Computed stresses. Compression.	Tested value. Compression.
Large arch ring	1:2½:5 concrete	498 lbs./sq. in.	*4,140 lbs./sq. in.
Abutment piers	1:3:6 concrete	370 "	
Small piers	1:4:8 concrete	370 "	
Foundations	1:5:9 concrete	270 "	
Soil	hard clay	8.2 tons/sq. ft.	
Steel hinges	cast steel	22,750 lbs./sq. in.	78,200 lbs./sq. in.
Concrete backing for hinges	1:2:2 concrete	938 "	†6,830 lbs./sq. in.

*28-day test. †90-day test.

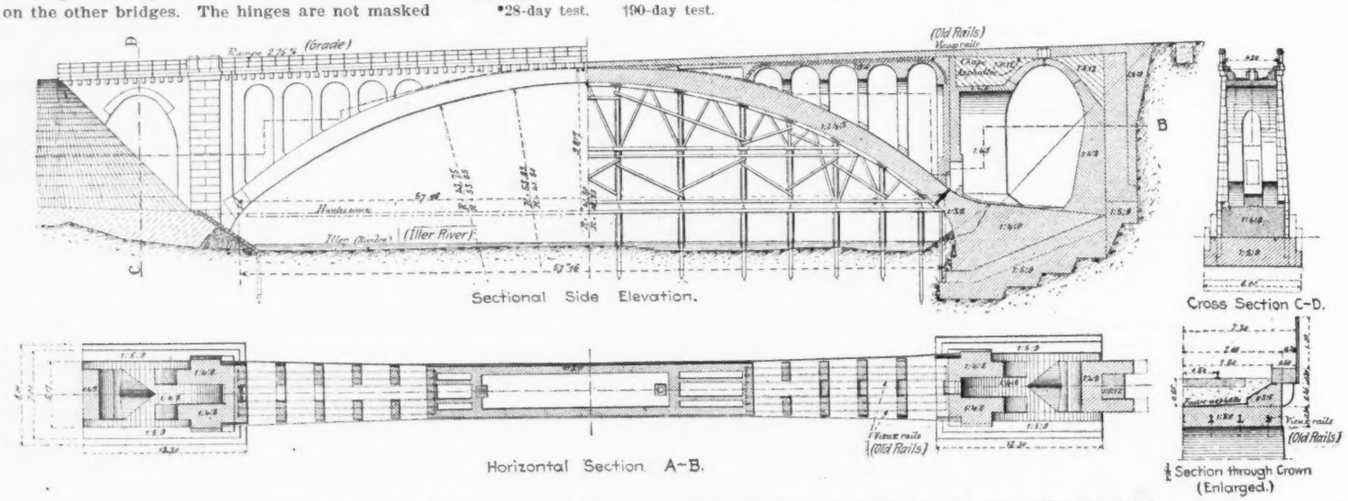


FIG. 2. ELEVATIONS AND SECTIONS OF THE THREE-HINGED CONCRETE ARCH AT LAUTRACH, BAVARIA.

and the vertical open joint immediately above the haunch hinge is in full view. Except for such difference and some minor details shown on the drawings, in construction and design this bridge is like the two others which will be more completely described.

THE KEMPTEN BRIDGES.

The Kempten bridges each consist of one large arch and three smaller arches. One is for four tracks with a maximum roadway of 54.0 ft. and the other for two tracks with a maximum width of 25.0 ft., the roadway narrowing at the middle the same as on the Lautrach Bridge (see Fig. 2). The wider of the two bridges is made of sets of two separate and parallel arches with a distance of 4 ins. between (Fig. 6). The main arches are of the basket handle type, with a total span between abutment piers of 211.6 ft. and a rise of

Data on the material and stresses on various parts of the bridge are given in the accompanying table.

The very high compression test shown by the concrete in which the steel hinges were bedded (6,830 lbs. per sq. in. in 90-day test) should be noted. This concrete was made of 1 part cement, 2 parts sand and 2 parts basalt grit. The mixture in the foundations (1:5:9) is interesting in this country, where so lean a mixture is very rarely used in any important work, even in foundations.

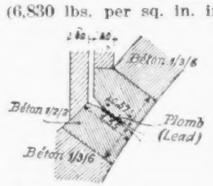


Fig. 8. Lead Hinge Joint of Small Arch, Kempten Bridge, Bavaria.

CENTERING.—The centering of the large arch of the Lautrach Bridge presents no new features, but that of the main arches of the Kempten bridges is interesting, particularly in its similarity to the centering of the Walnut Lane arch at Philadelphia, described in Engineering News Jan. 31, 1907. The main staging of the falsework is of structural steel resting on temporary concrete piers which extend for the entire width of the bridge. On the steel staging and separated from it by powerful screw jacks, which are used to strike the centering, rests the timber falsework. The part of the piers extending up the intrados of the arch toward the hinge is supported by a trussed steel frame (Fig. 4). As in the Walnut Lane Bridge the entire centering is used to erect one-half of the four-track bridge and then moved intact to the other half. For

FIG. 3. A CONCRETE BRIDGE WITH THREE-HINGED ARCHES AT KEMPTEN, BAVARIA. SPAN, 211.6 FT.; RISE, 86.7 FT.

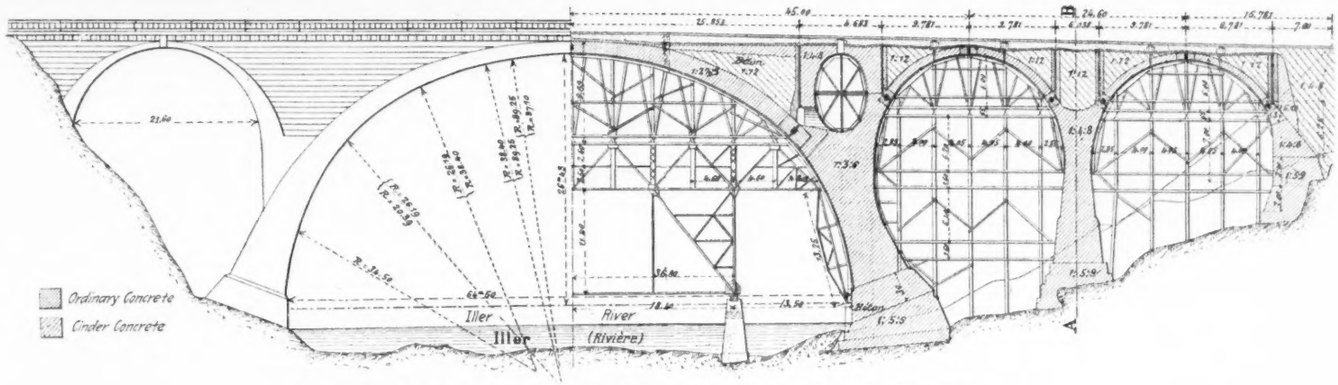


FIG. 4. ELEVATION OF THREE-HINGED CONCRETE ARCHES AT KEMPTEN, BAVARIA.

the two-track bridge the centering is taken down and re-erected. The first project was to move the whole falsework from one bridge to the other, but was judged to be too hazardous, as the moving of this mass of 275 tons involved a translation of 200 ft., a rotation of 30° and a rise of 6 ft. On account of an accident on the falsework of a bridge at Munich some little time previous to the erection of these bridges, the government ordered a high factor of safety to be provided in the design of the centering for the arch.

No untoward incident or cracks marred the construction of the large arches, the work upon which was carried out with great care. The striking of the centers, which was started in the middle of each semi-arch and worked symmetrically each way toward the hinges, took about 15 days. The large arches were allowed to set about six weeks before dismantling was commenced, and the smaller ones about 15 days.

It should be noted that all dimensions given on the cuts accompanying this article are in the metric system.

VOLUME OF AIR REQUIRED FOR DRYING PURPOSES.*

In the drying of green lumber fresh from the water the air supply per pound of water evaporated frequently ranges between 2,000 and 5,000 cu. ft. A great deal depends upon the manner of exposure to the air, i. e., to the method of piling in the case of lumber to afford a considerable amount of ventilation. The enormous volume of air actually required for the mechanical process of taking up and carrying away the water is the best evidence of the necessity of positive and ample air movement. Not only through the drying house, but also in and around the material treated.

Commercial success in drying, therefore, requires that the substance be exposed to the air in the most efficient manner; that the temperature of the air be as high as the substance will stand without injury and that the air change or movement be as rapid as is consistent with economical installation and operation. Under the mistaken idea that heat is the only essential to the drying process, dry rooms are only too frequently built with ample heating surface, but with practically no opportunity for air circulation. Only because of natural leakage of air do they avoid dismal failure. Even where provision is made for circulation of air it is often the case that dependence is placed solely upon the effect of the heat to produce air movement. Such movement is sluggish at best and frequently uncertain. High stacks tend to improve the circulation, but in so far as the heat required to operate them might be otherwise usefully employed, they are expensive as compared with mechanical means. Hence the use of the fan, in connection with which all of the heating surface is massed in a remarkably small space. With the fan the volume is always under control, as is also the temperature.

The process of drying is only an exaggerated case of heating in which the temperature to be maintained greatly exceeds that required in a well ventilated room. A supply of air sufficient to change the volume once in 8 to 15 minutes in the ordinary structure is all that is necessary to meet the conditions of good heating and ventilation, but a supply equivalent to a complete change in the free spaces within the dry room 60 or more times per hour is required in the lumber dry kiln. The temperature therein will range up to 150° or more. Owing to its greatly increased absorptive power the air at this temperature greedily takes up and carries away the moisture in the exposed surfaces.

The method of application and the temperature to be maintained depend entirely upon the character of the material to be dried. Glue and gelatine require low temperatures and ample volume. Knit goods in the piece are dried by internal application of heated air under pressure which distends the piece, the air escaping through the meshes. Cotton, wool and the like are spread upon beds formed by covering with netting enclosed boxes into which the hot air is discharged and from which it can only escape through the material above.

*From data furnished by the B. F. Sturtevant Co., Boston.

THE CANADIAN HYDRO-ELECTRIC POWER COMmission has received offers for supplying power from the Canadian Niagara Development Co., The Electrical Development Co. and the Ontario Power Co. Requests for contracts for this energy have been received by the commission, from the Western Ontario Niagara Power Union of Municipalities.

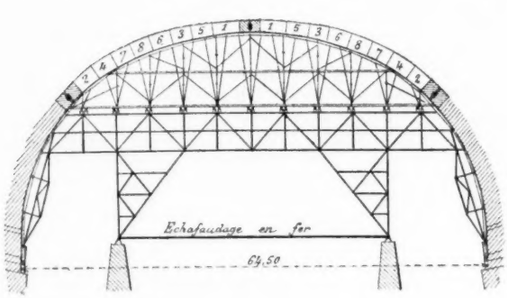
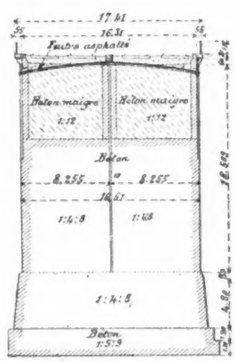


Fig. 5. Method of Laying Arch Ring, Kempten Bridge.



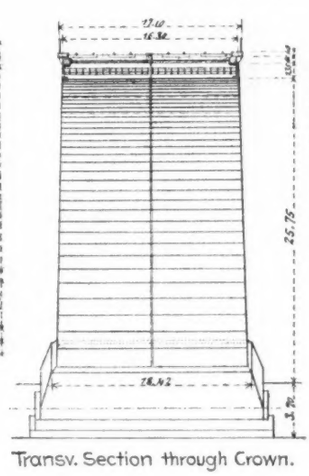


Fig. 6. Sections Through Crown of Main Arch and Through Intermediate Piers, Kempten Bridge.

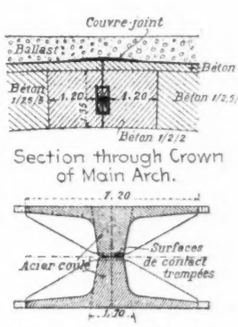


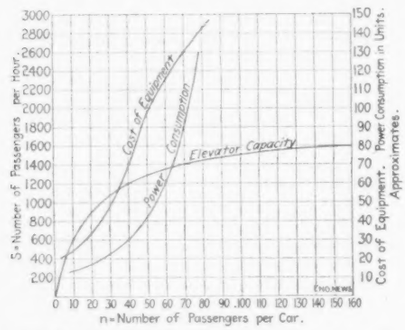
Fig. 7. Crown Hinge, Three-Hinged Concrete Arch, Kempten, Bavaria.

THE COMPARATIVE CAPACITY OF ELEVATORS AND ESCALATORS FOR HANDLING CROWDS.

By CHAS. D. SEEBERGER.*

Even among engineers who are specialists in elevator design and installation, there is a wide divergence of opinion as to the relative capacity of various types of elevators. Two square feet per passenger is the generally accepted unit of car area, and power equipment is based on that assumption. It is, of course, possible to pack people more closely if done without regard to comfort—not to say decency—and celerity of the service. In stores where the elevator service is closely supervised, there is usually a limit placed on the number of passengers per car appreciably greater than two square feet per passenger, while in actual experience, as a result of the desire to be near the door, disinclination to occupy the corners, and the requirements for prompt dispatching a battery of elevators, the space allotment is raised to about 3 sq. ft. Between the theoretical 2 sq. ft. and the practical 3 sq. ft., there is a loss of nearly one-third the rated capacity.

As the actual time in transit where short rises are involved is but a small fraction of the trip, the question of speed of the elevator car is relatively of minor importance, and the determining factors are the time consumed by passengers in entering and leaving the car, in the opening and closing of gates, and the incidental delays, and



Curves Showing the Relation Between Car Capacity, Traffic Capacity, Cost of Equipment and Power Consumption for Standard Passenger Elevators.

to these factors we must look to determine the number of trips per hour. The losses at gates and incidentals are practically constant, while the time for entering and leaving is a variable depending upon the number of passengers, consequently a hard and fast rule cannot be followed in determining the number of trips hourly.

Let T = total time of a trip in seconds,
t = time in actual transit in seconds,
H = hour = 3600 seconds,
R = round trips per hour,
n = number of passengers per trip,
S = sum of passengers per hour,
k = constant from actual observations of time per passenger spent in entering and leaving, gate works, etc.

Then $T = t + n\,k$

$$R = \frac{H}{T}$$

and $S = R\,n$

The value of k varies according to the character of the service from about 1.9 sec. at an underground station, rush hour, largely men; to 3½ sec. in a dry goods store, women and children. While the hourly rate is universally employed, a smaller unit is preferable. A minute rate would more readily answer the requirements of the engineer dealing with trains on minutes headway; indeed the plotted curves of elevator service in stores show peaks of short duration, for which the service should be properly proportioned.

The curve shown herewith is based on an actual installation of large elevators in public service in which the values of t (= 40 seconds) and k (= 2 seconds) are known. The car, however, is regarded as of extensible area accommodating n passengers as shown on the x axis.

It will be observed that the curve practically conforms to the parabola equation; and, when read in connection with the curves representing cost of equipment and power consumption, that for such service, the maximum of advantage is reached at say 40 passengers per car; beyond this point a duplication of equipment is to be preferred. To illustrate: The 40-passenger car (read on the left) has an hourly capacity of 1200, while the 50-passenger car reads 1280, an increase of about 7% capacity, but the prime cost of the equipment (read on the right) is 70 for the 40-passenger car to 98 for the 50-passenger car or 40% more, and the power consumption (also read on the right) is 30 against 43, or 43% greater. The cost curve, be it understood, represents the approximate cost of the stock sizes of apparatus adapted to the various loads, while the power curve approximates their rated consumption.

It is, of course, understood that in this calculation we have assumed a maximum duty, a continuous supply of passengers and an unbroken continuity of theoretical trips, a condition, it is needless to say, which is not met with in actual practice. The fact is also emphasized that the curve pertains to a given situation where the values of t and k are known, it is, however, applicable in a general way to elevators located at underground railway stations and depots generally.

It is the purpose of this article to suggest to the engineering fraternity, a standardization exact enough for all practical purposes, by which the intermittent elevator service and the constant escalator service may be compared, namely, the *total square feet of platform presented at a landing per hour*. An example will best illustrate the subject. Assume that an elevator with a platform 8x12 ft. or 96 sq. ft. makes 25 trips per hour.

$$96 \text{ ft.} \times 25 = 2{,}400 \text{ sq. ft.}$$

An escalator presents 4,000 steps of 6 sq. ft. each at the landing hourly $4{,}000 \times 6 = 24{,}000$ sq. ft., or 10 times the elevator service; or conversely, the same service in one-tenth the time.

Or, reduced to terms of the minute instead of the hour, we have

Elevator: 40 sq. ft. of platform per minute.
Escalator: 400 sq. ft. of platform per minute.

Or, again in terms of passengers on the basis of 2 sq. ft. per passenger:

Elevator: 20 passengers per minute.
Escalator: 200 passengers per minute.

It is possible to crowd the steps of an escalator to the same, or even greater degree than the elevator platform, owing to the marshaling effect of the individual steps. In point of fact there are escalators in daily use which, for short intervals, carry as many as five persons to a step, though it cannot be regarded as good service, therefore whatever the divisor, whether 2 sq. ft. or more, or less, according to the opinion of the engineer, it is equally applicable to elevator or escalator and consequently the *platform area per unit of time* affords a perfect standard for comparison.

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LICENSING ARCHITECTS has been practised for about ten years in Illinois. On Dec. 1, 1906, 704 licenses were in force. Of these, 501 were granted without examination to architects in practice at the time the law was passed, and 203 were granted to applicants who passed examinations. Altogether, exactly 700 licenses were granted without examination within six months after the licensing act became effective. The last report of the Board of Examiners of Architects states that

> the fact that only 501 of these [licenses] are now in force demonstrates that 199 [licensees] have either died, removed from the State, or given up the profession. The Board has every reason to know that most of them are in the latter category, and that their failure to succeed has been due mainly to the fact that they were unfitted for their professional work. This number almost exactly balances the number of new men licensed, all of whom are believed to be competent. The actual number of practising architects remains about the same as it was ten years ago. Can anything show better than these figures the elevation of the profession through the establishment of the examination and licensing system?

Professor N. Clifford Ricker, of Urbana, is President of the Board, and Mr. Peter B. Wight, Chamber of Commerce, Chicago, is Secretary.

THE INSPECTION AND MAINTENANCE OF BRIDGES AND ALLIED STRUCTURES.

The subject of maintenance engineering is one which has had a growing importance as a systematized study. Especially has this been so in the preservation of bridges and allied structures and in the last few years numerous papers have appeared which embody the result of the experience of men whose work lies in this direction. One of the latest of these papers was presented by Mr. Herman K. Higgins, Assoc. M. Am. Soc. C. E., before the Boston Society of Engineers on Jan. 23, 1907, and has just been printed in the "Journal of the Association of Engineering Societies." A portion of the paper with some of the discussion follows:

The writer was for several years in the employ of a railroad company, and part of his duties consisted in the examination of structures for the purpose of determining, first, when they should be replaced; second, what loads they should be allowed to carry in the meantime. Some of the conclusions he has arrived at may be pertinent and possibly useful.

Many of us are accustomed to consider bridges as classified, (a) by material—stone, concrete, iron, wood, etc.—and (b) by use—Highway, railway, etc. In the olden time the classification by use was characteristic—highway bridge practice being materially different from railway practice. Since the advent of interurban street railway and motor vehicles the distinction has largely disappeared, and in many respects the two classes are now practically one. Classification by material is still to a great extent characteristic, although the lines of demarcation are becoming much less sharply defined. The reinforced arch, for example, is neither like a stone arch, which it resembles in appearance, nor like a plate girder nor a continuous column, which it resembles (somewhat) in stress analysis.

CLASSIFICATION OF BRIDGES.

The writer has in recent years thought of bridges as classified thus:

1. Stone—voussoir arch, including brick, and block concrete. 2. Concrete—monolithic, hinged or not, reinforced or not. 3. Iron or steel, with floors of any material. 4. Wooden, with only part of tension members of metal and with bolts, spikes, etc., of iron or steel.

These four classes call for somewhat different treatment in detail, the general considerations remaining the same for all.

The determination of the proper time for renewal of a structure ("structures" being a term to cover many things besides bridges) is a much more complex problem than is often supposed. It depends upon design, condition, present loads, probable future loads, maximum vs. ruling loads, and last, but not least, on many economic considerations.

In order to judge intelligently and get the full life of structures a comprehensive system of inspection with reports and permanent records is necessary. Many structures likely to be condemned under irregular inspection would be retained under a regular system of examination and recorded reports. It will readily be seen that in the nature of things this must be so. If the inspector knows that he will see the structure again in six months or a year, he can work much closer to the full useful life of his material. If he does not know when the structure is to be next examined, he will, properly, condemn everything that falls below a high standard even when he knows it is good for one or possibly three years but not longer. A proper system of records also allows him to devote an extra proportion of attention to older structures. There is a chance for much waste right here in the absence of system.

Stone bridges, also brick, were, not so very long since, thought to be permanent. The older arches are, under present train and highway loads, often in need of careful watching; fortunately they always give notice of overstraining, and intelligent supervision will often hold them in service long after the stresses approach the elastic limit, indicated by cracking and spalling. These structures were designed long ago for loads then absurdly heavy but now outgrown; and loads are still growing. It may be that an occasional excessive load may be tolerated, whereas the ruling load must always be within safe limits to provide for the appreciable though unmeasurable effect of fatigue of material. It frequently is the case that very inexpensive repairs—concrete lining, buttresses and buttress arches of brick or concrete, iron tie rods, etc.—will suffice to make the structure perfectly safe and secure for many years more, especially when economic conditions make entire reconstruction inadvisable.

Block concrete (voussoir) arches are not often met with, but compare in all respects, except unit stresses, with stone arches.

Concrete, arches, monolithic, reinforced or not, are of comparatively recent date and have not yet begun to approach their limit of life. They differ from stone

in that internal conditions cannot so readily be judged by external signs. It is going to be a very ticklish proposition to know just when to condemn such bridges. What knowledge we have of them is based almost entirely on laboratory tests, and the few service tests available indicate the need of much caution. The writer does not consider it safe to approach anywhere near as close to the elastic limit as good practice warrants with either of the other types of bridges. By the time these structures approach their limit of usefulness, many, if not most, of the records pertaining to their original construction will have been lost. It is so with many old bridges built so recently that the writer can readily recall their preliminaries, and will, no doubt, continue to be the case. Roads built by promoters rarely have complete records of structures. With the original records missing, it will be impossible to correctly compute stresses, and the inspector will have no easy problem to solve. An iron or wooden bridge can be measured, a stone arch can at least be approximated, but to measure the iron in a Monier or Melan bridge will require a divining rod that will make the witch hazel of olden days looks very old-fashioned. Even in case the records are available the inspector's path is by no means lined with roses. The effect of vibration with modern loads is not well known nor is the quality of the concrete used. In a couple of generations this problem may be historical only, but the writer believes that most of the concrete structures of this generation will have become historical also. We need permanent records and recorded and published experience. We shall continue to build concrete structures and we greatly appreciate the experimental data prepared for us by so many eminent members of this and other societies. We could do little without it; nevertheless, we do not know nearly as much about concrete as we do about iron or stone, and as bridge inspectors we shall feel suspicious of every crack and stain, however harmless, till we can find out what caused it.

SPECIAL NOTES ON STEEL BRIDGES.—Iron or steel structures nearly always present a comparatively easy problem. If original plans are missing, they can usually be obtained from the builder, whose enterprise in advertising (by name plate) often fills a serious blank in records. The condition of the iron is apparent on proper examination. It may help some one to mention some of the locations of incipient failure. In old structures the rivets and splices usually show earliest signs of distress. Plate girders and built up beams need special attention to web rivets near ends; the hammer will not find the trouble; white paint before the last rain is much better but often not attainable; there is often some sign of motion of flange angles on webs. If motion exists, the structure should be watched for increase of motion. If it does not increase, there is usually no cause for alarm. When it does, measures should be taken to stop it. Lateral and sway connection are often weak and work loose; often a few rivets a size larger and some reaming of holes will suffice as a remedy. Roller bearings that do not roll are the rule for old bridges and are important only on long spans. If badly rusted, plate girders should be examined for holes in webs near corners adjacent to flange angles.

In truss bridges one should look for "upholstered" struts, rust streaked plates near and below riveted joints, weak details of tension members and laterals, pins subject to severe bending moment; old pins were often small and members were often put on at variance from plans. Details near bearings on masonry are often subject to excessive corrosion.

Sometimes abutments and parapets move and "pinch" the structure. This applies to all bridges—iron and wood.

Truss pins should always have cotters outside the nuts. Old bridges usually do not. Nuts may be keyed on with machine screws. One should look for missing nuts and pins working out, also for loose rivets at floor connections, also for evidence of wear of pin at foot of hip verticals.

When the structure is known to agree with the plans, the stresses may be computed. For old structures the writer believes the best method involves the old-fashioned "wheel loads." He prefers and uses the diagram of moments in the older editions (1890) of DuBois, "Strains in Framed Structures," checked by comparison with older computations. This diagram is sufficiently elastic to use when all or few members are in question; it is little work to make a diagram for each class of locomotives as it appears on the road; it gives as close approximations as are warranted by the indeterminancy of load distribution. The writer uses graphic analysis and "equivalent loads" on design of new structures, but believes that for old structures the so-called analytical determination is preferable.

Unit stresses must be kept below the elastic limit even for the most occasional loads and after liberal allowance for impact has been made. In determining allowable unit stresses, knowledge of the composition and molecular structure of the metal will often provide data of great value. It is often possible to get samples for analysis and micro-examination. This must, of course, be done intelligently to be of value, and sound judgment is needed to determine when the expense of such testing will be warranted. A soft, fibrous, homogeneous material can safely be allowed to carry loads close to its probable elastic limit, whereas a crystalline, hard or streaky specimen should have a good margin. Sulphur and phosphorus in any considerable quantity subject a specimen to serious suspicion. Manganese, on the contrary, is an element of safety if not grossly in excess.

Rivets are the most unsatisfactory and indeterminate element in the structure. Only one who saw them driven can properly judge them. The hammer and touch test is useful but not certain. The writer has seen ⅝-in. rivets put into ⅞-in. holes and headed to give perfect appearing heads, and they were tight. The hammer was defeated as a test. The inspector was not known to the gang and happened to see the trick done. Most of the rivets in the truss in question were planned of the small size so the danger of non-discovery was real. Many, if not most, old bridges were built without inspection. What wonder that we need to examine rivets with care. The writer has also seen (not on his work) rivets drawn down to enter a badly matched hole. With full size heads both sides, this fault is undiscoverable. After all tests are applied it is, with old bridges, a matter of faith and careful examination for evidence of movement in the joint. It will surely move before it falls and proper inspection will detect danger before it becomes imminent. Rivets rarely fail in shear. They are safe in most cases up to unit stress seven-tenths of tensile elastic limit or even more, if shop work is first class. The bearing or crippling unit stress can run much higher, nearly to the ultimate strength of the iron. The writer knows of bridge girders in service up to within a few years, the rivets in which, figured by the usual methods, were carrying pressures 25% in excess of the probable ultimate compressive strength of the iron. The shop work was extra good, iron well fitted, and friction well developed. There never was the slightest sign of distress. In such cases the writer has always considered high bearing stress on rivets rather a suspicious than a positive indication. Such structures should be watched.

BRIDGE FLOORS.—The floors of iron and steel bridges need as careful examination as the main girders. The old way (still current) of using two layers of plank on highway bridges is pernicious in many places. The lower plank decays, the upper wears or decays. On city bridges, which require a new layer of wearing plank every six months or oftener, the double plank is economical and safe, as the frequent renewals of wearing plank allow effective inspection. On town and country bridges the upper layer decays rather than wears, and the writer has usually found the lower plank in decidedly unsatisfactory condition, entirely unfit to discharge its proper function, the carrying of the loads to the stringers. The writer considers all such floors subject to suspicion and has usually recommended their replacement with a single layer of plank usually thicker than the original bottom layer. Four-inch plank is heavy enough in most cases and leaves the floor determinate.

WOODEN BRIDGES.—Wooden bridges often surprise us by their capacity for carrying overloads, not safely, but nevertheless successfully in many instances.

The construction of electric railways often proves the limiting feature of wooden highway bridges, and it may well happen that the distribution of material and the moving of machinery imposes a greater strain on the bridge than the operation of the cars. These loads being rare and the cars frequent, it may still prove to be the cars that make necessary the renewal of the bridge.

City bridges are more often of iron than country bridges and are subject to corrosion in concealed places; country bridges when of iron often lack paint and corrode in exposed places.

Wooden stringer bridges usually show most decay in the middle of tops of stringers. The writer considers such decay usually not a cause for renewal until it extends far enough to cause spreading or local crushing of the stringer. This principle does not hold for concentrated loads or bearings on wall plates. These must be sound. Timber protected from weather is subject to a form of deterioration quite distinct from decay. Builders would say, it has lost its "life," it becomes soft and punky, the grain does not separate properly. Such timber should be replaced. Timbers of different species often destroy each other. A Howe truss with pine or spruce posts (struts) and oak angle blocks has been known to retain a mere shell of sound timber in the braces, the interior near the ends being like cheese. Only the auger intelligently used will detect this form of decay. Truss chords near oak clamps and keys will bear close examination. A chord failure usually means business for the wrecking train. Counters sometimes break short off; they need attention, especially when old.

There are often economic reasons, and political as well, which finally prove most important factors in the determination of time of replacement. With highways the political predominates, with railways usually the economic.

LOAD SCHEDULES.—It is commonly but incorrectly believed that railways keep their structures up to the requirements of their heaviest rolling stock. As most of our railways, in the East, at least, were built some time ago, it will readily be seen that there must be many antiquated structures to limit the allowed traffic. The better the structure originally was, the more likely it is to be or become a limiting feature of the road.

Most railroads, if not all, publish a schedule for the use of freight shipping agents, showing allowed loads for each line between junctions or important stations. Any load offered in excess of these allowed loads is either reloaded to give lower wheel loads, or sent by a slightly differing route. This means that for each line between such points there may be one or more structures the strength of which limits the safe load. In case the line has been in existence for some years it usually is the case that there are several such structures, or in case the line was built by promoters, as is often the case, all the structures are equally light. It may well be that they are as good as new, only weak. If the traffic on such line is not too heavy or if the line is not part of a main trunk line, it will manifestly not pay to rebuild one or all the bridges, but the loads will be limited. Of course there is also a similar schedule for operating and motive power officials, showing limits for locomotives. If, on the other hand, a single bridge limits the traffic, it becomes purely an economic problem to work out the advantage to be gained by its replacement.

In case the completion of a new connecting line opens up a new trunk line, or even an alternative line to be used as a trunk line in case of wreck or emergency, it may be the wisest course to replace the many structures. It may happen not to be a bridge that limits the line, but a turntable at a critical point, an ash pit, a roundhouse, a station with covered train shed; any of these or other conditions may limit the weight or size of locomotives and render unwise, that is, uneconomical, the replacement of other structures, weak but sound and in good condition and sufficient for the traffic as limited.

On branch lines it is nearly always an economic problem and may be political as well. Perhaps some much desired industry needs facilities for handling heavy loads. The near expiration of a lease may render it very unprofitable to rebuild a structure, even at the alternative of excessive repair charges.

On main lines it is sometimes needful to run heavy trains for advertising purposes even at an economic loss. Many structures are rebuilt for this reason long before weaker structures on branch lines are even strengthened.

It will thus be seen that economic or business or even political considerations may take precedence of strictly engineering or constructive problems, in determining time of replacement of structures.

After replacement is determined upon there is still a chance for economic errors of design which will make trouble later. A line good for only say 120,000 lbs., on account of structures in first-class condition but of weak design, would not be entitled to 200,000 lb. structures, unless the latter are to outlive the former, and unless the future cost of strengthening will considerably exceed the present cost, enough to pay the interest at least. A structure near the end of a weak division adjacent to one stronger may well be made equal to the stronger division. In any case the economic problem should be worked out.

A paper on this same subject has been presented at the 1906 September meeting of the American Society of Civil Engineers and abstracted in Engineering News, August 23, 1906, in which the author, Mr. W. J. Watson, Bridge Engineer of the Osborn Engineering Co., covers similar ground to the above. An editorial in Engineering News, in September 13, 1906, also attacks the same problem. It will be seen that the present writer has ideas somewhat at variance with those expressed by Mr. Watson and the editor. This is, no doubt partly due to a different set of conditions, and partly to the inevitable difference of opinion between different individuals. Where such differences exist the writer sees no occasion to change his former conclusions.

DISCUSSION.

In the discussion which followed the paper Mr. J. P. Snow, M. Am. Soc. C. E., said, in part:

In addition to the paper by Mr. Watson, cited by the author, the following may be named: A paper by A. J. Himes, in Bulletin 80, October, 1906, American Railway Engineering and Maintenance of Way Association, which contains references to several papers and articles besides that of Mr. Watson; a paper by Wm. Marriott on strengthening early iron bridges, and the Austrian Railway Ministerial Order of August 28, 1904, concerning new and old bridges, both of which are in Bulletin 4, April, 1906, vol. 20 of the International Railway Congress; and a paper by J. E. Greiner on the "Life of Iron Railroad Bridges," in Trans. Am. Soc. C. E., vol. 34, page 294. A paper by Mr. C. D. Purdon in the Journal of Engineering Societies, vol. 33, page 325, gives a method of classifying locomotives as to their effect on bridges, together with a sample schedule of bridges, which shows at a glance the approximate maximum

strain on every bridge under any locomotive that has been classified.

I agree entirely with the author that much depends on field examination in determining whether a bridge should be retained, strengthened or rebuilt for existing traffic. A table for each structure, showing the maximum unit strain in its various parts, should be in hand when such examinations are made. Rivets especially are to be judged almost wholly by their action in service. Much has been written about allowable shear and bearing strains, dependence upon friction, etc.; and, of course, high units are proper signals for examination, but if they keep tight they can be trusted. As the author says, they rarely fail by shear; they will show their weakness by becoming loose long before they will fail.

Timber bridges show distress much more plainly than those of metal, but the latter can be so examined as to show signs of weakness long before failure need be feared under existing traffic.

The principal difficulty that confronts an engineer responsible for bridges arises from the desire to increase the loading on old bridges. In this case he must judge what will be probably reasonably safe from the computed unit strains. He has much to guide him by a study of the structures that show weakness under existing loads, but no fixed rule can be assumed, because no two structures behave exactly alike. My experience has brought me to the conclusions expressed in this and the following paragraph. They are the basis of my practice in handling old bridges, but I should not wish others to follow these rules literally, because in every case judgment based on the action of the structure in question under load must temper the said rules.

Pin trusses built 30 years ago will not bear the unit strains that riveted trusses of the same age will; and of the latter, a pony truss will bear higher strains than a deep through truss. This latter is contrary to the teaching of many engineers, but experience proves it to my mind. Plate girders merit the confidence that all repose in them; but riveted pony trusses behave, with me, just as satisfactorily. The only way that I have known plate girders to fail has been by the top flange getting loose on the web in deck structures. This is due to wheel loads rather than to flange strain; they become loose near the center of span as generally as at the ends. The allowed unit strain may well be 25% higher in the one type of bridge named above than in the other.

Within limits, also quite short spans, say below 20 ft., may be allowed to carry higher strains than longer ones. This with loads figured as static. I realize that this is another case of heresy, but I am sure it is true. The reason may be that the frame of the engine assists the bridge, or that the running rail helps it out, but the fact remains that at 5 ft. any old stringer will do, even if it figures above the breaking strain.

Our author does not fix upon an allowable unit strain. It is delicate ground, but without some basis for discussion no definite result can be reached. The unit strain and the impact allowance, as it is generally called, are balanced factors, and no intelligent consideration of the matter can be reached without treating them simultaneously. Again, in making impact allowance, the question of speed must be taken into account. Ordinary railroad traffic at speeds below 20 miles per hr. may be considered as static. At 60 miles the allowable unit should be decreased 25% or the load increased 33%, if a constant unit is used, on ordinary types of 100 ft. span and less. With these features in mind, the ordinary units used in designing may be doubled before a bridge need be condemned for high strains. This is a broad rule; its application needs to be varied to fit each individual case.

There is a wide difference of opinion among laymen, and quite a little among engineers, as to whether a bridge is suitable for given loads. This difference covers all the ground between not exceeding the load for which the bridge was designed and the idea that this load can be multiplied by the ratio of the ultimate strength to the unit strain used in the design (the so-called factor of safety) before the bridge will fail. Both these views are manifestly extreme. The first would seem to claim that if bridges will carry safely more load than that for which they were designed they are thereby proved to be unnecessarily heavy. We sometimes hear it said that they are purposely made over-strong to provide for future increase of loads, but if we ask just how much overload is provided for the answer is indefinite. If we expect a certain overload we ought to take that load as our weight to start the design with.

The fact is we do not design our structures to just carry the prescribed load with safety and not break down, but to carry it satisfactorily; and every increase of the prescribed load will decrease the satisfactory working of the bridge. The vibration and deflection will increase under increased loads and more frequent repairs will be called for until the load is such that the strains in first class riveted work are about twice what were used in the design, when the action of the bridge will generally become so unsatisfactory that its renewal will be welcomed by all. Multiplying the original load by the so-called factor of safety cannot be done on account of unequal distribution of strains, imperfect material and rough workmanship.

Of course deterioration is a live factor in condemning bridges. Modern structural steel corrodes with alarming rapidity. Why it should corrode so much faster than wrought iron is being studied by a Committee of the American Society for Testing Materials. The question of molecular change (crystallization) is not talked about so much now as formerly. If badly overstrained, say about three times the allowable unit, iron will get brittle; but most of the brittle material found accidentally in service was just as brittle when put in as when found.

The celebrated paper by Mr. Watson, referred to by the author, advocates the Pritchard formula for impact when investigating old structures. A comparison of this formula with others is given in the annual proceedings of the American Railway Engineering and Maintenance of Way Association, Vol. 6 (1905), page 257. That comparison shows that for ordinary tie-floor single-track railroad bridges there is but little difference in the final results between the Pritchard, the Schneider and the Cain formulas. For double-track trusses and floor beams the Schneider formula gives more reasonable results than the others, while for ballast floor bridges the opposite may be true.

These formulas were designed for use in proportioning new bridges and they provide very liberally for the impact of moving trains as well as for secondary strains due to non-symmetrical connections. If applied in analyzing old bridges a unit strain of at least 25,000 lbs. per sq. in. should be used and then many short bridges will be ruled out that are entirely adequate for the desired service. It must be rememebered that these formulas provide for speeds of 60 miles per hour or over, and if slower speeds obtain, the impact addition may be less. I prefer computations on a static basis and unit strains varying from 15,000 to 20,000 lbs. per sq. in., according to judgment.

Timber stringers, if fairly sound, will not fail at 2,400 lbs. per sq. in. fiber strain. In tension or compression, however, timber will not generally safely carry this strain. In this case it becomes a matter of side support and end connections which in timber trusses are generally very faulty.

Concerning the deterioration of timber, the author's remarks accord with my experience. A feature not touched upon by him is the fact of greater durability of timber in cold climates than in warm. Altitude is equivalent to higher latitude in this respect; and the difference in the life of exposed timber is very marked between the latitudes of Boston and Montpelier, and between the altitude of Boston and Ashburnham, Mass. The difference in the life of Southern long-leaf pine is fully three years at the points named.

In a letter to the Society Mr. Joseph R. Worcester, M. Am. Soc. C. E., said:

Mr. Higgins states that unit strains in metallic structures should never be allowed to reach the elastic limit of the metal. In this there can be no doubt that all engineers will agree, but are we to assume that, if the strains do not reach the elastic limit, he would pass the structure as safe? In other words, would he allow a tensile strain in a main member of steel of 30,000 lbs. per sq. in.?

Mr. J. E. Greiner, assistant chief engineer of the Baltimore & Ohio R. R., in a discussion of the subject for the Committee on Iron and Steel Structures of the American Railway Engineering and Maintenance of Way Association, has recently presented a convincing argument against allowing any higher strains in old structures, in which there exist uncertainties as to the nature of material as well as perhaps imperfectly designed details, than would be allowed in new structures, properly designed and built of carefully inspected material. As against this, of course, we may say that in designing new structures we advisedly use lower strains than we know to be safe, for the very purpose of providing for an unknown future increase of live load. It is, however, an illogical method of making this provision. When modern specifications are tending more and more toward the method of reducing all possible strains, including impact, to an equivalent static strain, and using for the unit strain a flat unit supposed to be correct for a static load, we should logically, if we want to provide for an increase in future loading, make the desired provision by an increase in the assumed live load.

The general practice at the present time, both for railway and highway bridges, as well as for other structures, is to use a unit strain of about one-half the elastic limit of the material. This factor is considered correct in places where the load assumed is an absolute maximum, as, for instance, where it consists of a definitely determined dead load only. There is reason enough for allowing this factor on account of possible irregularity in the metal and internal stresses due to imperfection of details. Can it be said that these elements are less likely to be present in old structures than in new? Manifestly not.

The only excuse for allowing this factor to be reduced in an old structure would be that by careful watching it might be safe to rely upon distress becoming apparent before an accident could result. This would seem to be a doubtful dependence unless an almost daily careful inspection were made, which would be well-nigh impossible.

The author tells us that the hammer is not a sure guide in the detection of overstrained rivets. The writer agrees so far as to admit that no examination of a structure without calculation of strains is as safe as careful analysis. At the same time the writer has always held the opinion that a rivet which was really tight under the hammer was doing its work, and that it would give evidence of weakness, by becoming loose, before it would fail. It is undoubtedly true that many rivets, if not all, do their work solely by clamping the parts together so strongly that motion is prevented by friction, even though the shank does not bear on either side. This condition is perfectly normal, and so long as the friction is sufficient, is ideal. The time will come, in the writer's opinion, when rivets will be figured by friction, which bears a constant relation to the shearing strength, rather than by bearing and shear, which never are called into play until the joint slips to some extent. Even when the joint has slipped, and the bearing and shear come into action, if the rivet remains tight, it cannot have been seriously overstrained. If the author has seen cases where rivets ringing tight have failed in service, it would be instructive if he would describe the circumstances.

The author's fears with regard to the future of reinforced concrete bridges do not appear to be well-grounded if we may judge by analogy with reinforced concrete beams and slabs. In one respect, at least, there is an element of safety in all concrete structures not existing in stone and brick, that is, the growth in strength that occurs in this material with age. It seems probable that where the determining strain is pure compression, this increase in strength ought to go far towards taking care of the increase in live loads for a very considerable period. On the other hand, the addition of a tensile strength at the critical points of an arch, even though the exact amount or nature of the reinforcement is uncertain, should contribute a certain ductility above the elastic limit not possessed by voussoir masonry. It is well known that beams or slabs of reinforced concrete will often deflect to one-thirtieth or one-twentieth of their span before failure, while cracks may open almost enough to allow the reinforcement to be measured. These circumstances certainly ought not to be considered as adding anything to the hazard.

Prof. George F. Swain, M. Am. Soc. C. E., said in part:

In judging of old structures, the engineer will be assisted by remembering two facts: First, that the engineer is aided by nature; and second, that many things help support a structure which are not allowed for in the usual computations.

With reference to the first of these, a structure will stand up if it is a physical possibility for it to do so; that is to say, if a condition of stress is possible which is consistent with the physical conditions and the conditions of equilibrium. The yielding of one piece will throw the stress elsewhere. Even the failure of a piece usually considered necessary does not always mean the destruction of the structure.

With reference to the second point: In short wooden stringer spans, for instance, while we consider the load to be carried by the stringers under the rails in reality there are, in addition, as a rule, guard timbers, two track rails, frequently two guard rails and frequently side stringers. Many short spans would stand with perfect safety if the stringers under the rails were entirely removed. Then, again, friction is an element which frequently helps. In wooden trusses it is often the factor which really causes the truss to stand up. In riveted structures the friction on the rivets, usually neglected, may be sufficient to carry the entire stress on the rivets. The writer does not, however, agree with Mr. Worcester in believing that it would be proper to allow for this friction, since it is an essentially uncertain quantity, while the strength of the rivet is a positive quantity. Again, we allow for the nominal diameter of the rivet while in reality the rivet is about ⅛-in. larger. With a ⅞-in. rivet, this alone gives an increase in strength of about 15%. Another practice of many engineers in not allowing for the action of the web of a plate girder in carrying the moment is an incorrect assumption on the safe side, like many others frequently made. In all of these ways many structures are really much stronger than they are computed to be.

It has been the writer's practice to allow the stress in the main members of a structure, making in some cases some allowance for impact, to reach about three-fourths of the elastic limit before considering the structure to have reached the danger line. No definite rule, however, can be laid down, as varying circumstances must be taken into account.

The writer believes, in the case of steel structures, that the principal cause of renewals is defects in design

and not overstraining of the main parts. The loads to be carried by railroad bridges have increased very much in the past 30 years and yet there are many well-designed railroad bridges built 30 years ago which are still in existence and carrying these increased loads with perfect safety. On the other hand, some structures 15 years old or less have had to be renewed on account of defects in design, although the increase in weight of rolling stock has been comparatively small. A great deal of money has been wasted for our railroad corporations by improper design.

Coming to specific defects, the writer would like to say a few words in favor of the reliability of hard pine as a structural material. He has always found that after some years' service hard pine looks much worse than it is. The sap decays, but the interior of the stick not more than ¼-in. to 1 in. from the outside is generally perfectly sound. White pine, on the contrary, he has not infrequently found unreliable and subject to dry rot. Many, if not most, of the wooden trusses built 30 to 50 years ago, in this part of the country at least, were built of white pine. The writer remembers one occasion in which he discovered by the use of the auger that three out of four sticks in a lower chord of such a truss were mere shells. In the examination of wooden trusses he has found that while the auger is, of course, the most useful instrument, the sound under a hammer, and even the color of the wood, are sometimes valuable and reliable indications.

The writer knows of no reliable means of ascertaining whether the vertical members of a Howe truss are equally stressed, unless they happen to be of equal size and of the same free length, in which case the sound under the hammer will be a good indication. If the sizes of the bars in one member are different, or if they have not the same free length, the sound will, of course, be no indication at all. The writer believes that such bars are often very unequally stressed.

Mr. L. S. Cowles said:

The author's idea of judging the fitness of an old bridge from a common-sense point of view rather than a too theoretical one seems to be the sanest method to pursue. However, the mere keeping of unit stresses below the elastic limit would hardly warrant a feeling of contentment for the inspecting engineer unless the stress were well below, or say under three-quarters, of the elastic limit.

With regard to the indeterminate element of rivets in a structure, I feel that if a rivet is tight under the hammer test, it must be doing some of its intended work, even though it does not fill the hole, or may otherwise lack the requirements of a so-called first-class rivet.

The most frequent overloading of city bridges seems to come from the ever-increasing weight of the electric car, whether urban or interurban. The main girders and trusses are no doubt less likely to be overstrained than the floor system, and especially the connections of stringers to floor beams. In old city bridges the practice of supporting wooden stringers carrying car tracks on single unsupported shelf angles is to be deplored, and all such defects should be remedied by means of vertical stiffeners under the outstanding leg of angle, or where there is not sufficient space the angle may be supported by vertical bolts and a yoke over the top chord of floor beam.

The question as to what constitutes a "liberal allowance" for impact for electric-car loading is a much mooted one. Mr. C. C. Schneider, in his very excellent paper on "Bridges for Electric Railways," "Street Railway Journal," issue of Sept. 22, 1906, introduces his impact formula for railroad bridges in a modified form, so that the addition for impact due to electric car loads is practically one-half that due to locomotive loads. This seems perfectly safe, as the speed maintained on most electric railways is much lower than on trunk lines, and the rotating effects of the motors, is no doubt, less disastrous than the reciprocating motion of the steam locomotive parts. Certain defects common in the track system of many electric roads might tend to equal the pounding effect of poorly balanced locomotive driving wheels, but over such a roadbed high speeds could not with safety be maintained.

Mr. Frederic H. Fay, M. Am. Soc. C. E., finished the discussion as follows:

The speaker indorses, in the main, the author's views in regard to reinforced concrete arches and believes that his criticisms are applicable to reinforced concrete structures in general. While the strength of an existing structure of wood or iron may be determined within reasonable limits from no other data than those obtained from a field examination, in the case of the structure of reinforced concrete, with the original records missing, it will be extremely difficult, if not impossible, to determine its strength by inspection. The increase in live loads should be taken into account when one is considering the question of building a bridge of steel or of reinforced concrete; and it should be remembered that while, in many cases, a steel structure can be built which will admit of future strengthening at slight expense, thereby prolonging its life, it is by no means a simple matter to strengthen a reinforced concrete structure. Because of this difficulty in strengthening, the uncertainties of workmanship and the practical impossibility of inspection in service without the original data, not to mention our present lack of knowledge of the behavior of reinforced concrete structures during a long term of years, the speaker believes that good designing calls for a considerably higher factor of safety in reinforced concrete structures than is used to-day in structures built of steel.

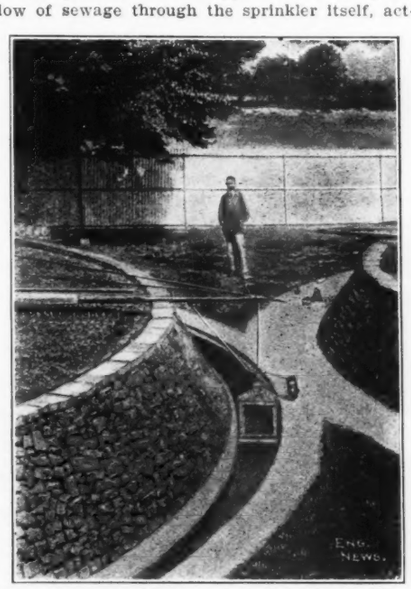
At the present time there is a remarkably rapid increase in live loads on highway bridges carrying street cars. In Boston, for example, a number of city bridges originally designed to carry a uniform live load of 100 lbs. per sq. ft., or a single 20-ton wagon, are now being strengthened by the Boston Elevated Railway Company to take trolley cars weighing 50 tons.

The speaker takes exception to the author's statements as to the advantages of a single layer of plank over two layers in the floors of highway bridges. Where one layer of floor planking is used it will usually be allowed to continue in service until it is worn nearly through or it actually breaks under some heavy load; in short, until it becomes dangerous. With two layers of plank, the lower layer should be designed for strength and the upper for wear; and even if the upper layer be worn entirely through, the strength of the lower planking remains to carry the load to the stringers. The author admits that on city bridges "the double plank is economical and safe, as the frequent renewals of wearing plank allow effective inspection." The speaker does not agree with the author's statement that on town and country bridges the lower layer of plank will rot out before the upper layer has become so unfit for service as to require renewal. The speaker is of the opinion that in all highway bridges with plank floor-surfacing, whether the bridges be for city or for country use, it is the better practice to use two layers of planking.

In the case of plank floor bridges over railroads, it is often desirable to provide floor planking which will require complete renewal at frequent intervals, for usually it is only when the planking is so renewed that the metal work beneath the floor is repainted. In such bridges it has been, for years, the practice of the city of Boston to use untreated spruce for the lower layer of planking as well as for the upper layer, in order that the lower planking would be renewed and the metal structure repainted with reasonable frequency.

A NOVEL SPRINKLER MOTOR FOR THE PERCOLATING SEWAGE FILTER AT WELLINGTON, ENGLAND.

The number of different sewage sprinklers in use in England and the ingenuity shown in their design is remarkable. This is in line with the common British practice of using no end of patented or proprietary devices in municipal work, many of which vary only in some unimportant detail from some well-established general type. We illustrate herewith something quite out of the ordinary, however, in a motor for driving the revolving of arms of sewage sprinklers. Most of the sprinklers of this type are driven by the flow of sewage through the sprinkler itself, acting on the well-known Barker Mill principle, and familiar to every one who has seen a common revolving lawn sprinkler at work. In the sprinkler illustrated the effluent from the filter is utilized as a motive power, instead of the unfiltered sewage.



Driving a Rotary Sewage Sprinkler by the Effluent from a Circular Percolating Filter at Wellington, England.

Our illustration is from the London "Surveyor" of Jan. 25, 1907. That journal states that sewage works have recently been completed for Wellington, Somerset, England, with Mr. Chas. J. Lomax, of Manchester, as designing engineer, at a cost of some $90,000. The works are described by our contemporary as follows:

> The sewage is treated on the bacterial system. The works consist of screening chambers, detritus tanks, open septic tanks, anaerobic beds, and six circular aerobic beds, each of 50 ft. diameter. These beds are provided with Mather & Platt's two-arm open trough rotating spreaders, the arms of which are fitted at their extremities with Howard's patent paddle attachment. By this arrangement of paddle attachment the spreaders are kept in constant work. No head is required, the power for working the arms being obtained from the flow of the effluent after passing through the circular beds into a circular channel.
>
> The apparatus is proof against wind pressure—a very important point—and will work as long as there is any effluent running. It has the additional merit of being inexpensive.
>
> The plant laid down at Wellington, which is giving every satisfaction, is capable of dealing with 450,000 gals. (540,000 U. S. gals.) (normal flow) in 24 hours.

No information is given regarding the fall in the circular effluent channel, nor time required to start a sprinkler after a period of idleness.

A NEW KJELLIN INDUCTION STEEL FURNACE has been put into operation at the Roechling Iron & Steel Works, Voelklingen, Germany. The capacity is 24 tons, 15 tons being poured at the end of a run, and 9 tons being left for the beginning of the next run. The power consumption is 736 KW. per hour. There are two smaller induction furnaces in operation at these works and a fourth in construction, having a capacity of 150 tons. This furnace will be used as a mixer and reservoir to keep molten metal, from the ordinary furnaces, to be run into the induction type for refining. A high production economy is claimed for the new plant.

TRACTION ENGINES FOR HAULING ORE are being employed in Idaho to handle the lead ore and concentrates from the plant of the Gilmore Lead Mining Co. to Dubois, on the Oregon Short Line. Hauling by teams for this distance of 85 miles cost $10 to $12 per ton (12 cts. to 14 cts. per ton-mile), and sufficient teams could not always be obtained. The traction engine is a three-wheeled, 110-HP. machine of the type built by the Best Mfg. Co., of San Leandro, Cal. It hauls four steel wagons of 15 tons capacity, and makes a round-trip in about four days; it runs night and day, being fitted with a powerful acetylene headlight. A wagon attached at the rear of the train provides accommodation for the crew not on duty. Each of the two crews consists of an engineman, fireman and helper. Coal is hauled from the railway and distributed in bins along the route, the engine consuming about four tons in 24 hours. There is tank capacity for a 15-mile trip, and water is obtainable at sufficient intervals. On the return trip there is one grade of 10% about ¾-mile long, leading to the mine. The engine can haul the empty train up this grade.

A SEWAGE TESTING DEVICE depending upon the electro-conductivity of sewage was described by Dr. S. Barwise before the Royal Sanitary Institute at Stafford, England, on Feb. 16 (Journal Royal Sanitary Institute, April, 1907.) Dr. Barwise stated that with the help of Dr. Dawson Turner, of Edinburgh, he found that with the

same quantity of sewage and effluent, the conductivity decreases with the purity effected.

The speaker may be quoted further, as follows:

> With his apparatus Dr. Dawson Turner found that the relative conductivity was somewhat as follows: Urine, 250 ohms; crude sewage of Chesterfield, 4,500; final filtrate of Chesterfield, 6,000; while the final filtrate at Buxton had a resistance of 11,300. The advantage of the method was that when it was known what degree of purity the various resistances at any particular sewage disposal works were equivalent to, all it would be necessary for the man in charge to do would be to read off his resistance and enter it up into a book as often as he was told. It would enable a local authority to know exactly what was taking place at their sewage disposal works. Conversely from the experiments he made, he had thought it was very likely that a plan could be devised for estimating the purity by using the effluent as an excitant of any ordinary voltaic battery. The more highly oxidized the effluent was, the more rapidly it would attack the negative pole. He found that by using copper and zinc electrodes with the Chesterfield sewage, he got a current with a voltage of .21, a tank effluent of .32, and a final filtrate of .38. Here again he brought this matter forward in the hope that it would start experimental work on new lines.

This was practically all that Dr. Barwise said on the subject, his remarks being incidental to a discussion of a paper by Dr. George Reid.

AUDIBLE CAB SIGNALS AS DISTANT SIGNALS ON THE GREAT WESTERN RY., ENGLAND.

The Great Western Ry. of England has lately installed a system of electrically-controlled audible cab signals on a branch line about 22 miles long, extending from Whitney to Fairford in Gloucestershire. The principal feature of the system is the replacing of the semaphore distant visual signal by audible signal apparatus located in the cab of the locomotive, consisting of a steam whistle and an electric bell. The former gives the "caution" signal, while the latter gives the "clear" signal, and either, when once set in operation, continues to sound until shut off by the engine-driver.

Both signals are operated by means of a short stationary contact rail, located between the regular rails at each signaling point, and a contact shoe attached to the locomotive. The "caution" signal is actuated by the interruption of a local electric circuit on the locomotive, effected by the lifting of the contact shoe by the contact rail. The "clear" signal is caused to operate by the contact shoe picking up a current from the contact rail, the apparatus being so arranged that the bell not only rings, but the whistle is prevented from giving the "caution" signal. By this arrangement it becomes impossible for broken circuits, snow or ice, etc., to cause a "clear" signal to be given, as the "caution" signal will always blow if the local circuit is broken or the contact shoe fails to pick up a current. Failure to get either signal is a sign of danger.

The arrangement of electric circuits is shown in Fig. 1. The contact rail R is a T-bar mounted upon an insulated wooden base (Fig. 2), the main portion of the T-bar being 4 ins. above the level of the tops of the rails, while both its ends are at a lower elevation. The contact shoe L is fastened to the frame of the locomotive (Fig. 3) and insulated therefrom. The switch S is mechanically connected to the contact shoe, but is insulated both from it and the frame of the loco-

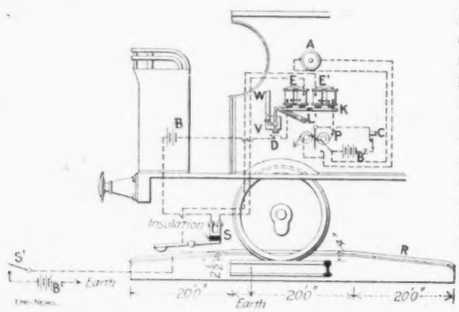


Fig. 1. Sketch Showing Arrangement of Electric Circuits for Giving Audible Signals to Engine Driver; Great Western Ry.

motive. B, B¹ and B² are batteries; S¹ switch at signal box; E and E¹ electro-magnets with armatures yoked together; K this compound armature; W the "caution" whistle and V its valve controlled by the armature K; L¹ a lever for resetting the "caution" apparatus; P a polarized relay; C a push button; A the "clear" bell; and D a switch.

In normal working condition an electric current flows from battery B, through switch S, through electro-magnet E, to switch D, thence back to the battery. It may be noted that D has no function in the working of the apparatus, it being simply a steam operated switch, which breaks the local circuit when the boiler pressure falls below 20 lbs., thus preventing waste of electric current, when the locomotive is not in running condition. The current flowing in this circuit energizes the electro-magnet E, which holds the compound armature K in place, and it in turn keeps the valve V of the whistle W closed. This may be called the "caution" circuit. A second circuit, which we will call the "clear" circuit includes battery B¹, switch S¹, contact rail R, contact shoe L, magnet E¹, polarized relay P, frame of locomotive, railway track and by ground return to battery. On passing over the

Fig. 2. A View of Track, Showing Contact Rail Between Regular Rails; Great Western Ry.

contact rail R, the shoe L is raised and the local "caution" circuit broken. This releases armature K, which in turn releases valve V and causes the "caution" whistle to blow. If, however, the towerman desires to give the engine-driver the "clear" signal he closes switch S¹, which, upon the contact shoe striking the contact rail, causes the current of battery B¹ to energize magnet E¹ and polarized relay P. This prevents the release of armature K, thus preventing the sounding of this caution signal, while at the same time the armature of the polarized relay closes a local circuit, consisting of battery B², push button C, polarized relay P and "clear" bell A. As shown in Fig. 1, this circuit will remain closed, as a portion of the current of battery B² passes through the second winding of the relay, and the bell will continue to ring until the engine-driver momentarily breaks the circuit by push button C.

The contact rail varies in length, from 40 ft. to 60 ft., the shorter one being used for branch lines and the other for main lines where high speed is maintained. The contact shoe is 7 ins. wide and has its face case-hardened. It is held in normal position by a strong spiral spring. When in this position it is 2½ ins. above the elevation of the tops of the rails, so that in passing over the contact rail it is raised 1½ ins. For testing the apparatus before leaving the yards, short contact rails are placed with their tops only 3 ins. above the elevation of the tops of the regular rails. From this arrangement it is seen the "caution" signal must sound when the contact shoe is raised only ½ in., or one-third its normal lift.

The line battery B¹ normally consists of 16 No. 2 Leclanché cells; battery B two large dry cells; and battery B² of four small dry cells. The whistle, bell, electro-magnets, polarized relay, resetting lever and push button have been put in one case, and this combined indicator located in the locomotive cab in easy reach of the driver.

The switch S¹ is controlled by the interlocking levers of the regular semaphore signals. In the case of single track roads, contact rails are placed on either side of the signal point, and in order that trains running in opposite directions may only receive signals from the contact rail intended for that direction, advantage is taken of the well known property of the polarized relay to operate with current flowing in one direction only. By referring to Fig. 1 it will be seen that in normal working condition the negative pole of battery B¹ is connected to the contact rail. On single track roads, however, the levers controlling the semaphore signals are so arranged that when a signal is set to give the right of way to a train going in one direction the contact rail controlling the audible signals for that direction is negatively electrified, while the contact rail for the opposite direction is positively electrified. In passing over this latter contact rail the electro-magnet E¹ will be energized, thus preventing the sounding of the "caution" whistle, while the polarized relay will fail to work as the cureent is flowing in the wrong direction, and, therefore, neither signal will sound.

The system was designed and installed under the direction of Mr. A. T. Blackall, Signal Engineer for the company. For the data from which this article was written, we are indebted to "Electrical Engineering."

AN INVESTIGATION OF CANAL LININGS is being made by the Irrigation Department of the Experiment Station of the University of California, under the direction of Mr. Elwood Mead, M. Am. Soc. C. E., Professor of Irrigation Institutions and Practice. The work was begun last year and is to be continued during the present summer. The object is to ascertain the cheapest material and simplest process for stopping seepage losses on both laterals and large canals. It is desired that the lining shall also prevent the growth of vegetation in canals and obstruct burrowing animals. We quote as follows from a circular letter issued by Professor Mead:

Cement is effective but costly. Clay for puddling is not to be had in many valleys where water is valuable. There are, however, an unusual number of raw materials in this state which promise well, crude oil and asphaltum being the most abundant, and the special purpose of this year's investigation is to test the native materials.

The results of last year's tests will soon be published as a station bulletin. They showed cement concrete to be most effective, but it cost six times as much as a lining of crude oil, which cut down the loss more than half, stopped all vegetable growth, and ended the work of gophers.

We wish the interest and cooperation of the canal managers and irrigators of the state. We are especially desirous of securing two things:

(1) From every irrigator or canal manager who has a lined ditch or reservoir, a description of how the work was done, what it cost, and what the result has been.

(2) Suggestions as to the experiments you would like to have us make, especially of material or methods of applying linings you would like to have us test.

Professor Mead informs us that he will be glad to receive information and suggestions from all parts of the country.

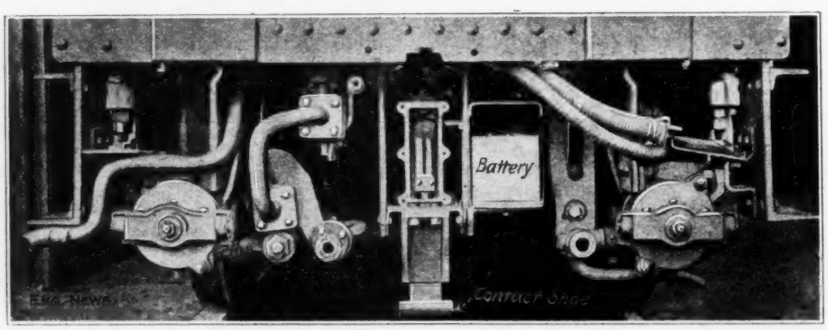


FIG. 3. A NEAR VIEW OF CONTACT SHOE ATTACHED TO LOCOMOTIVE; GREAT WESTERN RY.

ENGINEERING NEWS

A Journal of Civil, Mechanical, Mining and Electrical Engineering

Published every Thursday by

THE ENGINEERING NEWS PUBLISHING CO.

220 BROADWAY, NEW YORK

BRANCH OFFICES

CHICAGO: 1636 Monadnock Block
WASHINGTON, D. C.: Home Life Building
ATLANTA: Candler Building
RUDOLF MOSSE, BERLIN AND HAMBURG, GERMANY

GEORGE H. FROST, Founder

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Not long ago a pamphlet was published designed to establish the fact that the U. S. War Department had the power and was by duty bound to prevent the discharge of sewage into the Hudson River by the way of the proposed Bronx Valley Sewer. The arguments used were so sweeping that had they been valid the jurisdiction of the War Department would have been extended to every sewer outlet into navigable waters throughout the length and breadth of the United States, and nothing short of the whole engineer corps would suffice to exercise the suggested control. How little ground there was for that contention is strikingly shown by a letter from Acting Secretary of War Robert Shaw Noble, addressed to the Washington Heights Taxpayers' Association, a body that had petitioned the department to use "the power vested" in it to protect the Hudson from pollution. A portion of the letter follows:

> I beg to inform you that several similar protests have been received at the department, and upon reference of the same to the Chief of Engineers, U. S. Army, that officer reported that he knew of no federal law under which the pollution of the waters in question could be prevented, this being properly a matter for regulation by the state or municipal authorities, and that as, in his opinion, the only duty committed to the War Department is a consideration of the effect upon navigable channels, it was believed that as the injury to be expected to navigation is small and remote, the circumstances do not warrant intervention by this department at this time.
>
> With a view to ascertaining whether any warrant of law exists for interference by the War Department in the matter, the papers were referred to the Judge Advocate General of the Army, and after careful investigation he reached the conclusion that:
>
> Congress, in legislating to forbid the deposit of matter in the navigable waters, exempts from the operation of the prohibition matter which flows into such navigable waters from streets and sewers in a liquid state, and by so doing relegates authority to control such overflow to the constituted authorities of the state in which the drainage plans are executed.

The time may come when Congress will vest in the War Department or some other federal agency full control over the discharge of sewage into navigable waters. As to the wisdom of such a course we are not now concerned. What we wish to do is to call attention to the fact that the War Department now has no control over this subject. We also desire to emphasize anew the importance of paying careful attention to the legal and the engineering aspects of water pollution, if any success is to attend a campaign against it. The problem of protecting the natural waters of the country from pollution is altogether too complex to be solved out-of-hand by amateurs, no matter how zealous they may be. In the case of New York Bay and the Hudson River the utterances purporting to come from the Merchants' Association Committee have shown as little knowledge of engineering practicabilities and as little appreciation of the various grave questions of municipal policies and needs involved as of the authority of the War Department over the discharge of sewage into navigable waters.

One of the essential features of the plan for the municipal ownership of street railways by the city of Chicago was the issuance of street railway certificates which would be a lien on the street railway property, only, and not, like ordinary city bonds, on all the taxable value of the city. The law authorizing this plan, known as the Mueller law, was passed by the Illinois legislature in 1903. The certificate feature of the law has just been declared unconstitutional by the Supreme Court of Illinois.

Inasmuch as municipal ownership had been indefinitely postponed by the election which preceded this decision it matters little to Chicago for the present whether or not the proposed certificate plan was constitutional. Broadly speaking, however, the decision is of general interest, because the court held that such certificates as were proposed would be an addition to the bonded indebtedness of the city, and would thus increase the city's debt beyond the limit fixed by law. Presumably such certificates would be similarly regarded by the highest courts of other states. If so, then the Illinois decision will discourage attempts to further municipal ownership elsewhere by devices designed to escape debt limitations. In some states, however, constitutional bars of this sort do not exist, municipal debt limitations being under the control of the legislatures. In New York State, a constitutional amendment was secured, a few years ago, placing water bonds outside the debt limit.

It has been urged by both the friends and the foes of municipal ownership that all bonds issued to pay for revenue-producing municipal works should not be included in the city's debt as fixed by debt limitation requirements. The friends of municipal ownership urge that such a course is the only fair one, where a revenue is earned to meet interest charges and to redeem the bonds. It is said, for example, that the people of a city have in the long run to pay the interest on the cost of the plant which supplies them with water anyway, whether it be owned by the city or a private company. It is no greater burden on the taxpayer's property, therefore, to pay the interest on the bonds of a city-owned plant than on the bonds of a private plant, and hence these bonds should not be included in computing the debt limit any more in the one case than in the other.

The foes of municipal ownership, on the other hand, maintain that each municipal-service industry should stand on its own bottom, instead of having recourse to the general taxing power of the city for support. Otherwise, they urge, the patrons of an electric lighting plant, for instance, may be given low rates at the expense of the taxpayers as a whole. Underlying this argument is doubtless the idea that a change of this sort would make it far more difficult for cities to raise money for municipal enterprises. In other words, in the matter of capital-raising cities and private companies would be put more nearly on the same footing.

There is small likelihood that the Mueller certificate plan would have been copied frequently elsewhere, even if it had stood the test of the Illinois courts. No legislature, in the present state of public opinion, would adopt such a plan to defeat municipal ownership. The friends of municipal ownership would rarely find it advantageous, because if the legal debt limit of a city is reasonable bond buyers will look askance upon any means to get around it, and because the broader the security for municipal bonds, the better the market for them, and thus the more favorable the financial prospects of a properly-conducted municipal enterprise.

Bond buyers and taxpayers alike will always look askance upon great expansions of municipal indebtedness. If, however, our cities will once adopt a proper system of municipal accounting, then such of them as show that their public-service industries are really paying their own way, and never drawing upon the municipal treasury, will have no need for subterfuges to raise such capital as they may need for expanding their enterprises. True, low debt limits may stand in the way of extending municipal ownership, but it would then not be unreasonable to ask that such enterprises as can show that they are meeting all expenses, including capital charges, be placed outside the debt limit. Such a plan has already been proposed and has much to commend it—when accompanied by proper municipal accounting. It would often aid greatly in making each revenue-producing municipal enterprise quite independent of the municipal treasury, and still give the underlying bonds the benefit of the credit of the whole city.

The "future American highway," according to an inventor whose pamphlet we have received, will be a paved roadway 120 ft. in total width divided by longitudinal curbs into eight separate roadways, four for passage in each direction. He provides two 16-ft. roadways for animal traction vehicles and a 4-ft. walk at each side for the stray pedestrians who may still indulge in the antiquated method of locomotion that nature furnished. The rest of the width is devoted to automobile roads. As the cost of this remarkable highway would mount up to between $100,000 and $200,000 per mile, the inventor does well to call it a "highway of the future," and he might have added, of the dim and distant future.

TWO YEARS' OPERATIONS OF THE MUNICIPAL GARBAGE REDUCTION WORKS, CLEVELAND, O.

The paucity of cost data is nowhere more notable than in the field of garbage collection and disposal. It therefore affords us great pleasure to present, as we do below, some valuable figures on this branch of the municipal service. The figures are rendered all the more interesting because of their relation, as will be seen, to the municipal ownership question, and also because, superior as they are to most figures based on municipal accounts, they illustrate so forcibly some of the deficiencies and difficulties of municipal accounting.

Two years ago the city of Cleveland, O., bought the garbage reduction works and also the collection and shipping equipment of the private company that had served it for about six years. Since the purchase, the city has collected, shipped and treated its garbage. It thus holds the unique position of being the first and only city in the United States, and probably in the world, to own and operate a garbage reduction works. The twenty or more other garbage reduction plants in this country are all privately owned, and, for reasons that need not be considered here, their expenses and incomes are carefully guarded from the public eye. In contrast with this secrecy the operations of the Cleveland garbage department are set up by accountants* and published yearly. For copies of these reports we are indebted to Mr. W. J. Springborn, President of the Cleveland Board of Public Service. For the convenience of our readers, we have consolidated into one set of tables the physical and financial statements, some of which are for six-month periods, given in the two reports. Before presenting and commenting on these tables some general information on garbage disposal at Cleveland may be of interest.

In a letter to this journal written on April 19, 1907, Mr. Springborn stated:

The contract under which the works bought by the city were built was awarded in December, 1897, to the Buckeye Refuse Reduction Co., and sublet or transferred early in 1898 to the Newburgh Reduction Co., which was said to include virtually the same interests, and to have possessed a rendering plant for treating dead animals. The contract was to run for five years from the date the works should be put in opera-

*Ernst & Ernst, Public Accountants and Auditors, Schofield Building, Cleveland, O.

tion (which turned out to be Aug. 22, 1898). The contract price was $69,400 per year, in monthly instalments, regardless of the quantity of garbage treated. This price included both collection and final disposal.*

The garbage of Cleveland is removed from the houses in cans placed at their rear, collected in detachable steel wagon bodies, hauled to a railway siding at Canal St., Cleveland, loaded on cars, and shipped nine miles by rail to the reduction works, at Willow, O. From the cars the wagon bodies are lifted and moved to the sorting room by means of an electric traveling crane.

The reduction works were remodeled in 1900, under patents claimed by the Detroit Sanitary Works (Chamberlain process; see Eng. News, May 31, 1900, for description). There being uncertainty as to the renewal of the contract on its expiration, in 1903, and the continuation of the contract proving to be but temporary, the reduction works were badly run down when the city took possession of them on Jan. 1, 1905. This is indicated by the fact that although the inventory value of the collection, shipping and reduction equipment was $140,000, the actual purchase price was $87,500. Besides extensive repairs made in 1905, including the replacing of 5 of the 14 old digestors, the city contracted with the Edson Reduction Machinery Co., of Cleveland, for an enlargement of the plant.

In the old or Chamberlain system the garbage was cooked in digestors by means of steam. After running off the grease, the tankage was passed through hydraulic presses, run through an Anderson steam-jacketed dryer and finally screened and sold as a fertilizer base. The nominal capacity of the old works, at the time of purchase, was 100 tons a day.

The Edson plant, with a nominal capacity of 60 tons a day, was erected close by the old works. It was described for us by Mr. Springborn, under date of May 2, 1905, as follows:

> The digestors hold about five tons of the raw material. They are steam jacketed and also have a steam drum through the center, thus heating the material both from the in and outside. No live steam comes in contact with the garbage, thus avoiding the agitation which generally wears out the digestors. When the material is thoroughly cooked a valve at the bottom of the digestor is opened and the garbage dropped directly into a dryer, without exposing it to the air, thus doing away with the odor and usual unsanitary condition. When dried it is dropped upon a conveyor, passing over a screen, and is then sent to the percolating or extracting plant, where the grease is extracted by the use of naphtha.
>
> From tests conducted during the past two years we feel quite sure that not more than 1% of grease will remain in the tankage, and that the loss of naphtha will be slight. The tankage or fertilizer is of a much superior quality than under the old process, as none of the ingredients are wasted, owing to the fact that nothing escapes between the time the material is placed in the digestor and the time it is brought out in a finished condition.
>
> Suitable receiving tanks for night soil are being put in and connected with the digestors, so as either to pump or force the same by air pressure into the digestor without exposure to the atmosphere. We are confident that the night soil can be treated either in connection with the garbage or separately if desired, because we have conducted a number of tests which proved successful and demonstrated that there was a value to the material recovered.

*The reduction works were visited by a member of our editorial staff in October, 1898, and were described in Engineering News of May 31, 1900. Meanwhile the works were remodelled by Mr. E. S. Peck, Superintendent of the Newburgh Reduction Co. Mr. Peck supplied us with information that enabled us to bring our description down to the date of our article.

Table I.—Quantity of Garbage Delivered to Cleveland Reduction Plant for the Years 1904, 1905 and 1906.

	Pounds per month.		
	1904.	1905.	1906.
January	2,958,200	3,752,000	4,784,000
February	2,690,000	3,316,000	3,994,000
March	3,258,000	4,070,000	4,520,000
April	2,928,000	4,218,000	4,694,000
May	3,624,000	5,018,000	5,430,000
June	4,304,000	5,808,000	5,936,000
July	4,506,000	5,592,000	5,464,000
August	5,472,000	6,360,000	8,024,000
September	5,925,000	6,906,000	7,478,000
October	4,979,600	5,996,000	7,098,000
November	4,646,000	4,736,000	6,204,000
December	4,084,000	4,992,000	6,156,000
Total for year	49,374,800	60,764,000	69,782,000
Av. per month	4,114,567	5,063,667	5,815,166
Tons per year*	24,687	30,382	34,891
Tons per day (365 days).	68	83	96
Tons per day (312 days).	79	97	112
Population	426,000	437,000	448,000
Pounds per head per year	116	139	156

[*These and the figures that follow have been computed and added to those given by the auditors.—Ed.]

TABLE II.—CAPITAL ACCOUNT, GARBAGE COLLECTION, SHIPPING AND REDUCTION PLANTS, CLEVELAND, O., 1905 AND 1906.

[Rearranged and combined from the two reports of the auditors.]

	Operations in 1905.					Operations in 1906.				
	Original purchase.	Additions.	Total	Depreciation.	Final total.	Additions.	Total.	Depreciation.	Final total.	Total Investment.‡
50 acres land	$25,000	.....	$25,000	....	$25,000	.....	$25,000	....	$25,000	$25,000
Buildings	9,000	$3,443	12,443	....	12,443	$2,716	15,159	....	15,159	15,159
Machinery	30,000	3,052	33,052	*$3,154	29,898	12,117	42,015	*$3,919	38,096	45,169
Edson addition	....	†58,338	†58,338	....	†58,338	2,631	†60,969	*536	†60,433	60,969
Total reduction	$64,000	†$61,833	†$128,833	*$3,154	†$125,679	$17,464	†$143,143	*$4,455	†$138,688	$146,297
Collection and shipping equipment	23,500	7,704	31,204	*2,860	28,344	3,806	32,150	*3,351	28,799	35,010
Grand totals	$87,500	†$72,537	†$160,037	*$6,014	†$154,023	$21,270	†$175,293	*$7,806	†$167,487	$181,307

*Depreciation charged off every six months at rate of 10% per annum; it should be understood that this is a "bookkeeping item," merely.

†Not including $6,482, being 10% of the original contract price for additions made in 1905 by the Edson Reduction Machinery Co. This sum is reserved for alleged non-fulfilment of contract, and against it there remains a balance of $6,555, from a bond issue of $155,000, on account of the purchase and extension of the plants.

‡Without allowing for depreciation.

In a letter to this journal, written on April 19, 1907, Mr. Springborn stated:

> Since that letter [of May 2, 1905] was written we have had more than a year of actual experience in operating the Edson plant, and the results obtained have not been up to our expectations or in accordance with the tests previously made. In the first place, the amount of grease left in the Edson tankage averaged more than 3% instead of 1%. The loss of naphtha was a little over 7 gals. per ton of dried material. The tanks for receiving night soil were installed, but none of this material was treated during the year 1906, the reason for this being that it took the entire capacity of our works to treat the garbage, and we therefore could not spare the digesters to treat night soil.

Having outlined the history and character of the Cleveland garbage works we may now consider the figures on the operations of the works during 1905 and 1906, as given in the reports of the auditors. Tables I. to IV. are recombinations of the figures named, with some additional totals and averages. The chief object of the rearrangement and additions is to bring together the operations of both years. Tables V. and VI. include some figures of our own, based on or deduced from the original figures.

Table I. shows monthly garbage collections during the last year of private service and during the two years of municipal operation. The increase in the collections, over 1904, is notable: one-fifth the first year and two-fifths the second year. This increase under municipal operation is all the more remarkable in view of the fact that during the six full calendar years of the contract service the increase was from 22,379 to 24,687 tons, or only one-tenth.

Table II., made up from various figures scattered through the reports, shows the original purchase price and the various subsequent changes in capital account, owing to extensions to the plant and to depreciation. It will be seen that in two years the actual investment by the city has been more than doubled, but something like this was anticipated, in view of the condition of the plant at purchase, and the price was fixed accordingly. The 10% allowance for depreciation is unusual in municipal accounting. Since it applies to machinery, collection and shipping equipment, only, the rate is none too high, but on the whole it may be considered fair. The failure to charge off anything for buildings is at least partly excusable in view of the large sums expended for repairs and renewals (Table III.)

Table III. exhibits the amounts of grease, tankage, hides, etc., produced or sold at the reduction works, with the prices and total amounts for which they were sold or inventoried. These figures are given by six-month periods, since they could not readily be shown by full years without recomputing average prices, and since the fluctuations in price are of interest and possible value.

In the annual report for 1905 Mr. Springborn stated that the grease extracted that year was 2.63% and the tankage about 6% of the raw material. This was by the old process. He estimated that the new process would yield 3.75% of grease and 18% of tankage. The actual yield of grease in 1906 was 3.06%, but, as is stated further on, only a small part of the garbage was treated in the new or Edson plant. It will be noticed that in 1905 a large percentage of the tankage was pressed, only, and that in 1906 the gross weight of pressed tankage still exceeded that of the dried tankage. At our request Mr. Springborn has given us, under date of April 8, 1907, some valuable information on the production of pressed and dried tankage, including the relative moisture in each and the exact damage effected by the fire of 1905.* This is as follows:

> The fire of 1905 did not interfere with the collection or reduction of the garbage, but merely prevented the drying of the material after the grease had been pressed out of it. At that time we had but one dryer, and as it was destroyed by fire, we were unable, for a period of about six weeks, to dry any of the material. This accounts for the large amount of pressed tankage sold in 1905. This pressed tankage contains about 45% of moisture, while the dried tankage has about 2 or 3%, which accounts for the difference in price at which it is sold.
>
> During the year 1906 the city operated both the old and the new plants, the old plant treating 56,276,100 lbs. and the new 12,937,900 lbs. The dried tankage recovered from the old plant was 6.83%, and pressed tankage from the same process 3.43%. If the pressed tankage had been dried, the average of dried tankage would probably have been 8%. From the garbage treated in the new plant we recovered 20% of dried tankage. Since the new plant, however, treated only about 18½% of the total garbage collected, the total amount of tankage recovered is somewhat less than we estimated it would be.

In Table IV. there are given the itemized receipts and expenses of the whole undertaking. The income is nearly all derived from the reduction works, but the expenses are distributed between collection and shipping and reduction. The decrease in the revenue of the collection department is partly due to the annexation to Cleveland of outlying territory (? ? ?).

The "net profit" shown in Table IV. can be considered as such only for the purpose of making a general comparison between municipal and private ownership and operation. For this purpose, as the table shows, half of the "gross income" in 1905 and nearly half in 1906 is based on the proceeds of general taxation. Moreover, the interest on the investment is left quite out

*An item of $3,082, "repairs and renewals on account of fire," appears in the financial statement for 1905 (see Table IV.).

TABLE III.—SALES AND INVENTORY OF PRODUCTS OF CLEVELAND GARBAGE REDUCTION PLANT, FOR THE YEARS 1905 AND 1906.

[Combined in one statement, from the two reports of the auditors.]

	Jan. 1 to June 30, 1905.			July 1 to Dec. 31, 1905.			Total, 1905.	
Material.	Quantity.	Av. Price.	Amount.	Quantity.	Av. Price.	Amount.	Quantity.	Amount.
Grease	756,833 lbs.	$2.854 cwt.	$21,601	841,430 lbs.	$2.91 cwt.	$24,492	1,598,263 lbs.	$46,093
Dry tankage	788,973 lbs.	7.085 ton.	2,795	1,140,226 lbs.	7.02 ton.	4,003	1,929,199 lbs.	6,798
Pressed tankage	2,560,620 lbs.	2.451 ton.	3,138	3,792,290 lbs.	2.493 ton.	4,728	6,352,910 lbs.	7,866
Hair	66 lbs.	.20 lb.	13	100 lbs.	.20 lb.	20	166 lbs.	33
Tails	122	.30 ea.	37	99	.30 ea.	30	221	66
Hides	117	3.775 ea.	442	412	3.042 ea.	1,253	529	1,695
Sundries	...	.....	165	...	.....	182	...	347
	Jan. 1 to June 30, 1906.			July 1 to Dec. 31, 1906.			Total, 1906.	
Material.	Quantity.	Av. Price.	Amount.	Quantity.	Av. Price.	Amount.	Quantity.	Amount.
Grease	1,024,895 lbs.	$3.02 cwt.	$30,976	1,115,380 lbs.	$3.76 cwt.	$41,940	2,140,275 lbs.	$72,916
Dry tankage	3,048,610 lbs.	8.42 ton.	12,849	3,233,918 lbs.	8.487 ton.	13,724	6,282,528 lbs.	26,574
Pressed tankage	285,400 lbs.	2.926 ton.	418	2,029,940 lbs.	2.522 ton.	2,564	2,315,400 lbs.	2,981
Hair	78 lbs.	.20 lb.	16	437 lbs.	.20 lb.	87	515 lbs.	103
Tails	48	.313 ea.	14	151	.30 ea.	45	195	59
Hides	258	3.56 ea.	919	420	3.55 ea.	1,493	678	2,412

of the account, "as same is paid," the report for 1906 states, "by the city out of general funds."

For the convenience of our readers, we have condensed and restated, as Table V., the main items of Table IV., adding thereto interest at 4% (the actual rate) on $100,000 for the year 1905, and on the total bond issue ($155,000) for the year 1906. Our capital sum, in each case, is an arbitrary item, but in so far as it is in error it favors the garbage service. The city issued $155,000 of garbage bonds in 1903, Mr. Springborn informs us. Presumably the proceeds were used for other city purposes until the city took possession of the garbage works, Jan. 1, 1905; otherwise the garbage service should be charged for interest prior to that date. The auditor's accounts show that on July 1, 1905, the garbage department had made a capital outlay of more than $100,000, while on Dec. 31, 1905, this outlay was about $160,000. It therefore seems fair to debit the garbage service with interest charges on $100,000 for 1905. A reference to Table II. will show that with equal fairness interest on the total bond issue may be charged for the year 1906. Strictly speaking, perhaps some of the interest should be charged to capital account, particularly in 1905, but such an adjustment might not be easy, even by the excellent auditing department of Cleveland.

The result of the computations shown by Table V. is a "net cost" of garbage collection and disposal for the years 1905 and 1906 of $64,868 and $64,368, respectively, as compared with the $69,400 paid under the old contract. The material increases in the amount of garbage treated bring the costs per ton down to $2.13 for 1905 and $1.86 for 1906, as compared with the actual payment of $2.81 to the Newburgh Reduction Co. in 1904, and $3.10 in 1899. What the contract rate might have been under renewal, it would be idle to conjecture. According to statements in the Cleveland newspapers about the time the contract expired the city and the company considered a proposition to continue the work at the rate of $60,000 a year, but under what, if any, modifications, we cannot state. The increase of 40% in the quantity of garbage collected and treated by the city in two years, compared with the increase of only 10% during six years of private service, indicates that the company did not meet the needs of the city. Whether or not it would have done so under a longer-term contract, or a renewal of the old contract, is, like the probable cost of the service under a renewal of contract, a matter of conjecture.

The apparent financial results of garbage reduction alone, taking the garbage as delivered to the reduction works, are set forth in Table VI. This table is based on Tables II. and V., and, like the latter, includes interest charges. On the face of the table, reduction, alone, cost the city practically nothing in 1905, while in 1906 it yielded a profit of $13,123, or a little under 40 cts. per ton on the 34,891 tons of garbage treated that year.

Neither Tables V. or VI afford all the data needed to compare the effects of public and of private ownership upon the municipal treasury, since they contain no allowance for the redemption of the bonds. Mr. Springborn informs us that the bonds run for a period of 20 years. He **adds:**

> All bonds and the interest thereon are paid by the Sinking Fund Commission from moneys derived from a general tax for this purpose. On this account the garbage department has made no provision for the redemption of the bonds.

Nevertheless, by the change to public ownership and operation of the garbage works, the yearly tax rate of Cleveland is increased by say $5,200 a year to redeem its $155,000 of garbage bonds.*

Besides sinking fund requirements there should be charged to the garbage service something for general administrative expense, including salaries, office rent and minor items. What these charges should be, in the present state of municipal accounting, it would be difficult to say. They are at least partly offset, however, by the cost of supervision and inspection of garbage contractors under private operation, which has not been considered in the comparative figures already presented. It should also be noted that the garbage department "pays taxes the same as any private company," as Mr. Springborn states, and that it contributes materially to the interest received by the city "from depositories on daily cash balances."

Reverting to the subject of bond-redemption charges, it may be said that some students of municipal affairs maintain that in comparing the cost of public and private ownership it is unfair to charge to expenses both depreciation and sinking fund allowances. If a choice is to be made between the two is it not better to choose the one that involves an actual transaction, rather than a mere bookkeeping item—unless, indeed, the depreciation fund is actually carried as a reserve fund? In the present instance, the depreciation charges set up in the accounts are higher than our allowance for bond redemption. Therefore, taking cognizance of actual transactions, instead of fictitious ones, would improve the yearly showings for the garbage service. We do not say that this course is the preferable one of the two, but certainly it is far more in accord, than is the other one, with a system of municipal accounting which seldom recognizes anything but cash transactions. Even if both depreciation and sinking fund allowances be included in the total expenses the operations of the whole garbage service of Cleveland are a credit to the city, while the reduction works alone would show a small net cost in 1905, which would be more than offset by the net profit of 1906. The saving effected by the change to municipal ownership would still be praiseworthy, on taking cognizance of the increase in the quantity of garbage handled; and this without allowing for other possible improvements in the character of the service. That the service is highly satisfactory to the people of Cleveland seems to be shown by the fact that complaints for non-collection of garbage have averaged only about 30 a week of late, many of which have been from persons who had recently changed their residence or who had failed to provide suitable cans or to place the cans where the collectors could find them.

Moreover, we understand that the pay of the collectors has been raised from a rate of $1.75 to $2 per day, and of the laborers to from $1.60 to $1.75, and that the men at the reduction plant work in 8-hour instead of 12-hour shifts, as under private operation. What the ultimate effects of these more favorable conditions for workmen will be upon the cost of the service and upon the other wage-earners of the city need not be discussed at this time.

We think it is evident that our comments on municipal accounting have been made in a kindly spirit, and with a full appreciation of how far superior the Cleveland accounts under discussion are to municipal accounting in general. Rarely does municipal accounting aim to be more than an exhibit of mere cash transactions, designed to show, not how much a given service has cost, but merely what has become of the corresponding appropriation. Creditable as are the Cleveland garbage department accounts, we find in them no interest charges, simply because another department looks after the interest on bonded indebtedness. And for similar reasons the garbage department does not credit itself with interest on its contribution to cash balances in city depositories.

Accounting deficiencies of the kind just noted become apparent as soon as an intelligent attempt is made to find what a given municipal service really costs the taxpayers. This becomes still more evident when it is desired to make a fair comparison of the relative cost of municipal and private ownership, or to fix rea-

Table IV.—Income and Expenses, Garbage Collection and Reduction, Cleveland, O., for the Years 1905 and 1906.
[Combined in one statement, from the two reports of the auditors.]

	1905.	1906.
Gross Income.		
For sale of product	$57,335	$96,351
For inventory of product	5,563	8,694
For sale of raw material	559	354
For rents	130	127
For miscellaneous revenue of collection department, etc.	2,294	1,465
	$65,881	$106,990
For taxes	66,554	85,423
Total income	$132,435	$192,413
Expenses at Reduction Works.		
For labor at plant	$26,735	$43,732
For coal at plant	10,974	19,980
For superintendence and clerk hire	3,895	3,363
For repairs and renewals to plant	7,110	4,763
For press cloths	1,290	2,565
For insurance on building, etc.	314	288
For office supplies	124	146
For all other sundry expenses not itemized above: Oil, waste, light, telephone, lake water, etc.	1,921	5,113
For freight on product, purchase of dead animals, etc.	1,855	1,455
For press racks	231	806
For taxes	...	346
For commission, analysis, etc.	...	826
Total expenses at reduction works	$54,449	$83,384
Cost of Collection and Shipping.		
For labor, teamsters, etc.	$37,826	$43,829
For feed	9,168	10,991
For railroad freight on garbage	4,561	5,285
For superintendence and clerk hire	2,112	2,870
For shoeing, etc.	1,953	2,431
For harness repairs and renewals	784	1,204
For painting garbage cars, etc.	...	919
For repairs and renewals to freight cars owned, wagons, etc.	4,146	4,280
For rent Canal Street (including taxes)	559	473
For insurance	450	450
For all other sundry expenses not itemized above: Supplies for barn, telephone, oil, light, power, etc.	1,245	1,601
Total cost of collection and shipping	$62,804	$74,334
Extraordinary Expenses and Depreciation.		
Repairs and renewals on account of fire	$3,082	...
For auditing	150	$150
For loss of horses	250	1,473
For depreciatoin on old machinery and equipment at reduction works, at 10% per year*	3,154	3,382
Do., new, do.	...	536
For depreciation on wagons, harness, horses, stable and other collection and shipping equipment, at rate of 10% per year	2,860	3,351
Total extraordinary expenses and depreciation	$9,496	$8,892
Summary.		
Total income	$132,435	$192,413
Total expenses:		
Expenses at reduction works	$54,449	$83,384
Cost of collection and shipping	62,804	74,334
Extraordinary and depreciation	9,496	8,892
	$126,749	$166,610
Net profit*	5,685	25,803
	$132,435	$192,413

[*The report for 1906 states: "No interest charge on the bonds has been provided in the above figures, as same is paid by the city out of general funds." See Table V. for our recomputed figures, showing "net cost of service," instead of "net profit."—Ed.]

Table V.—Gross and Net Cost of Garbage Collection and Reduction at Cleveland, O., for the Years 1905 and 1906.
(Computed from the reports of the auditors.)

	1905.	1906.
Expenses, including depreciation	$126,749	$166,610
Interest on capital investment at 4%.	*4,000	†6,200
Total expense and interest	$130,749	$172,810
Gross income	65,881	106,990
Net cost garbage collection and disposal	‡$64,868	‡$65,820
Population	437,000	448,000
Net cost per capita per year	$0.145	$0.147
Tons of garbage	30,382	34,891
Net cost per ton	‡$213	‡$186

*Figured on $100,000; but total capital investment at the end of the year was $160,037, not allowing for depreciation.
†Figured, for simplicity, on total bond issue of $153,000; although the actual cost outlay at the close of 1906, not allowing for depreciation, was $181,307.
‡Under the five-year contract with Newburgh Reduction Co. the cost of collecting and treating 24,687 tons, in 1904, was $69,400, or $281 per ton. [In 1899, the first full calendar year of the contract, the amount of garbage (Eng. News, May 31, 1900) was 22,379 tons, giving a cost of $3.10 per ton.]

*Assuming that the sinking fund nets 4% a year, $33.58 per year per $1,000 would be required to redeem the bonds in 20 years, or $5,205 a year. See table by Mr. John W. Hill, M. Am. Soc. C. E., Eng. News, Jan. 25, 1894.

Table VI.—Revenue and Expenses of Cleveland Reduction Works, 1905 and 1906.

	1905.	1906.
Gross revenue, collection and reduction	$65,881	$106,990
Less revenue from collection department.	2,294	1,465
Revenue from reduction	$63,587	$105,525
Ordinary expenses	$54,449	$83,384
Extraordinary expenses:		
Fire repairs	3,082	....
Two-thirds auditing	100	100
Depreciation	3,154	3,918
Interest, 4%	*2,900	*5,000
	$63,745	$92,402
Add net profit, 1905	...	13,123
Deduct net cost, 1906	158	...
	$63,587	$105,525

*Interest bearing investment at the beginning of 1905 assumed as $74,000 and at the beginning of 1906 as $125,000.

sonable charges for water or light supplied by municipal plants.

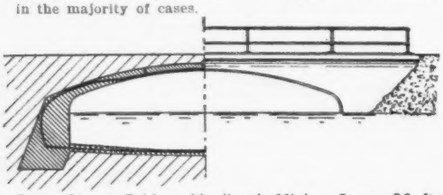
Thus far we have failed to mention one useful purpose to which the Cleveland figures may be put: a comparison of the cost of garbage reduction and cremation. Heretofore, such a comparison has been largely guesswork, since few American detailed figures on the actual cost of cremation and none on reduction were available to the public. The Cleveland accounts, as we have shown, separate the cost of reduction from that of collection and shipment. They seem to indicate that with a well-equipped reduction plant the final disposal of garbage may cost nothing, and may contribute slightly to reduce the cost of collection. But this is on the assumption that reduction works and furnaces would require equal haulage. Whether justly or not, public sentiment seems to require a reduction works to be located at a more remote point than a garbage crematory. If this be necessary, whether due to real or imaginary reasons, then it becomes important to determine the cost of the extra haulage to reduction works. On this account, it would have been helpful had Cleveland carried its admirable distribution of costs a little farther, as might have been easily done, so as to separate the cost of shipment by rail from the cost of collection and subsequent haulage to the railway shipping station. It seems probable that garbage furnaces might be located at the site of the railway shipping station, or at some other point equally near, if not even nearer, to the center of the city. If so, any net profit from garbage reduction might be more than offset by the extra charges for shipment to the more distant reduction works. However this may be in Cleveland, such considerations might be of weight elsewhere.

In conclusion, we trust that the Cleveland garbage figures may stimulate the officials of other cities to pay like attention to the cost of garbage collection and disposal, and that from garbage departments the influence of this good example may extend into other branches of the municipal service.

LETTERS TO THE EDITOR.

Is the Elastic Theory of the Arch Directly Applicable to Reinforced Concrete Arches?

Sir: There seems to be a growing tendency on the part of engineers to apply the elastic theory as developed by Greene, Howe and Cain to reinforced concrete arches, and to require stress sheets based on the same assumptions. The assumptions on which the above theories are founded are not always justified when applied to reinforced arches, and, in fact, do not apply with accuracy in the majority of cases.



Race Street Bridge, Ypsilanti, Mich. Span, 36 ft.; Rise, 5 ft.; Height of Opening, 10 ft. 6 ins.

I enclose you a half section of a reinforced concrete arch of 36 ft. span and 5 ft. rise, with 10 ft. height of opening, erected at Ypsilanti, Michigan, in 1901. If this arch be analyzed according to the elastic theory on the usual assumption that the end tangents are fixed in direction, the inner surface of the arch just above the springings will be found to be in tension, yet it must be apparent to any one at all familiar with arch action that such cannot be the case. The difficulty with the theory in this case lies in the fact that the abutments are elastic and in reality a part of the arch. The application of the elastic theory to reinforced concrete arches is, in nine cases out of ten, open to this same objection in a lesser degree. Very truly yours,

Daniel B. Luten,
President National Bridge Co.

Indianapolis, Ind., March 30, 1907.

A Method of Scaling Earthwork Quantities From a Profile.

Sir: In connection with my letter published in Engineering News, Feb. 21, 1907, it should be noted that the scale which I use is slightly different from the one published. The figures shown gave the cu. yds. per station for even feet of cutting, whereas the one in my use gives the heights of cutting for even hundreds of cu. yds. per station.

This enables the assistant to add the quantities mentally as fast as the estimator can read them from the profile. When a cut is finished, the figures show the total and saves the time of adding the quantities. They also show the total amount up to any station in the cut, and, if the quantities are read off in the right direction, this facilitates the segregation of quantities, giving a close and quick approximation on the overhaul, borrow and waste.

Referring to the letter of Mr. F. Lavis in your issue of March 28th, I prefer a slip with the zero either at the top or bottom instead of in the middle. A coat of varnish will keep it clean.

Formulas are quite useful in all tables that can be calculated by the method of second differences, as the latter are so readily obtained from an equation. Calculating by this method is very simple and quick, and an independent calculation of the final quantity is a sufficient check on the accuracy of the entire table.

In regard to the Greek letter, which gave offense, it had a distinct function. Perhaps it had something to do with transverse slope quantities. Very turly,

W. E. Davis.

Edmonton, Alta., March 21, 1907.

[A letter noting a similar change in the "quantity scale" has been received from Mr. G. D. Brooks, B. & O. R. R., Cumberland, Md.—Ed.]

Experiments on Material for Cold Springs Dam and for a Projected Dam Which Was Abandoned.

Sir: The writer has been particularly interested in the account, in your issue of March 7, of experiments made by D. C. Henny, Supervising Engineer, and E. G. Hopson, Assistant Supervising Engineer for Oregon, of the U. S. Reclamation Service. The writer inaugurated a similar series of tests at Spokane, Wash., for a similar purpose, during the winter of 1904-5, in connection with the Palouse Project. The project having been declared impracticable by a board of consulting engineers, of which Mr. Henny was a member, the experiments were discontinued. The Spokane experiments were conducted under pressures or heads ranging from 6 ins. to 122 ft., the object being to duplicate, as nearly as practicable, the conditions existing in high earth dams, and in the bottoms of reservoirs underlaid with porous materials below the surface soil.

The results of those tests showed that the pressure on the surface of the soil had a very marked effect upon the rate of percolation, through a diminishing rate with increase of time. It was also developed that almost the entire resistance to the flow of water through soils under considerable heads was at or near the surface, due to the collection of the fine particles of organic and inorganic substances filling the interstices in the soil. Even a very slight scratching or disturbing of the surface conditions seriously affects the rate of seepage. This hypothesis is also borne out by experiments made by the writer on very sandy soils in the vicinity of Priest Rapids, where with pure blow sand the rate of percolation decreased from about 15 to 5 cu. ft. per sec., due to a slight disturbance of the surface.

It was also observed in these experiments that the rate of seepage in cold weather was greater than in warm weather, whereas on the theory of the diminished viscosity of the water it would be less, which may be due to a lesser amount of organic matter in the water. It would seem, therefore, that the percolation deduced from experiments where a head of not more than 3 ft. was used, and where the effect of the action of the surface upon the flow had been partly eliminated by preventing a free discharge, would not be likely to give reliable estimates of the flow through a dam under 85-ft. head.

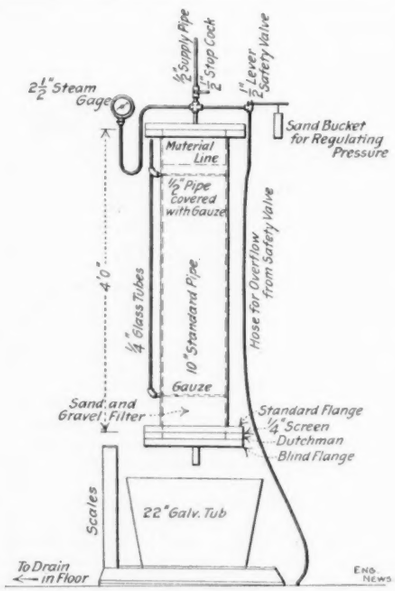
In connection with the Spokane experiments, three seepage tanks, identical in every respect except as to the depth of soil, were experimented upon to determine the effect of depth of soil on the rate of seepage. The results, as near as the writer can remember, showed very little difference in the rates of seepage for the three tanks, containing 3½ ft., 1½ ft. and ¾ ft. of soil.

The writer does not question the design of the Cold Springs Dam. It is based upon correct principles. But he does not believe that the experiments conducted by Messrs. Henny and Hopson indicate more than the relative merits of the materials experimented upon.

T. A. Noble, M. Am. Soc. C. E.

North Yakima, Wash., March 13, 1907.

[We append some further information regarding these tests, together with a sketch of the tank used by Mr. Noble—all based on reclamation service records. As the sketch shows, the tank was very much like a small pressure filter, and was quite unlike the open tanks used in the other tests. As is well known, pressure filters are soon clogged by the dirt retained on their surfaces and the material of which they are composed becomes packed. The bearing of these facts on the experiments, as well as of other



Sketch of Apparatus Used at Spokane, Wash., for Testing Material from Bottom of Proposed Reservoir on Palouse Project, U. S. Reclamation Service.

facts disclosed by the sketch, we leave to the judgment of our readers. We are informed that the chief object of the tests made by Mr. Noble was to determine the water tightness of the bottom of a proposed reservoir.—Ed.]

Patents on the Acid Treatment for Concrete Surfaces.

Sir: In the discussion called forth by the paper on the acid treatment of concrete surfaces, read by Mr. Linn White, Chief Engineer of the South Park System, of Chicago, before the convention of the National Cement Users' Association (Engineering News, Jan. 17, 1907), the use of muriatic acid has been suggested for the purpose intended. In regard to this suggestion I would call the attention of your readers to the fact that this use of muriatic acid is a direct infringement of the Black and Richards patent (No. 716,371) issued Dec. 23, 1902, under which patent all of the South Park work has been done.

Briefly stated, the specification of the patent relates that the invention consists in a process of so treating artificial stone (concrete) that the particles of sand, stone or granite or color pigment will project beyond the cement and so produce an appearance more closely resembling natural stone. In the main the specification reads as follows:

> The artificial stone may be made in any suitable manner and may consist in the usual ingredients (such, for instance, as sand or particles of limestone or granite united by cement). Pigments of any desired color may also be added to the composition to produce an artificial stone resembling natural stone of a color different from that of the sand or particles of stone used in the composition. For instance, red pigments may be added when it is desired to produce artificial red sandstone.
>
> After the composition has been molded and has hardened it is subjected to such a treatment as will remove the cement from around the superficial particles of the sand or stone used in the composition. A treatment which we have found in practice to be well adapted to produce the desired result is a bath consisting of a solution of acid (such, for instance, as five parts of water and one part of muriatic acid when the composition is at a temperature of about 80°). The acid attacks the cement and eats the same away from the surface of the artificial stone and also from around the superficial particles of sand or stone, so as to leave the latter projecting slightly beyond the cement, thereby rendering the cement less noticeable to the eye of an observer. After removing the artificial stone from the acid solution bath it may be rinsed off in clear water, so as to remove the acid solution from the surface of the artificial stone; after which, if desired, it may be immersed in a bath of neutralizing solution (such, for instance, as one pound of carbonate of soda in two gallons of water) in order to neutralize any acid which may remain on the surface of the stone. The stone may then be again rinsed off in water to remove the neutralizing solution.
>
> While we have referred to a solution of muriatic acid as being used to produce the desired effect upon the surface of the stone, we do not wish to limit ourselves thereto, as any treatment which will, by removing the cement around the particles of sand or stone, cause the latter to project beyond the cement is within the scope of our invention. Nor do we wish to limit ourselves to the particular neutralizing solution at all, as the mere removal of the artificial stone from the bath of the sand solution when the desired effect has been produced need not necessarily be followed by any other treatment in order to impart to the artificial stone the desired natural finish.
>
> CLAIMS.—1. The process of producing a natural finish on artificial stone composed of cement and particles of

natural stone which consists in removing the cement from around the superficial ingredient particles of the composition.

2. The process of treating which consists in subjecting the composition to a bath of acid solution.

3. The process of treating which consists in subjecting the composition to a bath of acid solution and subsubsequently to a bath of neutralizing solution.

4. The process of treating which consists in subjecting the composition to a solution of muriatic acid.

5. The process of treating which consists in subjecting the composition to a solution of muriatic acid and subsequently to abath of neutralizing solution.

6. The process which consists in subjecting the same first to a solution of muriatic acid and subsequently to a solution of carbonate soda.

7. In the art of manufacturing artificial stone, the process of producing a natural finish which consists in subjecting the stone when molded and dried to a solution of muriatic acid, then rinsing the same in water, then subjecting the same to a solution of carbonate of soda, and finally rinsing the same in water.

Thus it will be seen that the main point covered by the patent, the exposure on the surface of the grain and color of the aggregates by the removal of the superficial particles of cement, is safeguarded as to the use of muriatic acid, making use of that acid for the purpose intended a direct infringement of the patent.

As the process in question is one that is bound to have wide use in the treatment of concrete surfaces, it will doubtless be of interest to your readers to know something of the scope of the patent covering it, and the fact that its specifications cover directly and unequivocally the use of the acid in question.

Yours very respectfully,

J. K. Irvine,
Agent for the Patentees,

Sioux City, Iowa, April 22, 1907.

Examination Questions for Candidates for Position of Engineer in Charge of Works, Pocatello Water Co.

Sir: I have noticed at different times in Engineering News questions used in civil service examinations, but I have not seen published any of the forms used by some private corporations in determining the qualifications of applicants for engineering positions. It is, I believe, very unusual for private employers to require any kind of examination to test the engineering and general knowledge of applicants, such being involved only in the statement of experience and the list of references given by the applicant.

The enclosed form was evolved by the chief engineer and general manager of the Pocatello Water Co., Pocatello, Ida. Though bearing an ancient date, it is the form still used; at least I heard of its use only a year or two past.

On perusing this, those members of the profession who are not inclined to look on their chosen occupation as one calling for liberal culture, but as one wherein only the cold and soulless facts of material science are used as the working tools, should awake to the fact that extensive travel and the study of belles-lettres, as is customary with other gentlemen of means and leisure, is demanded.

After reading questions 3, 7, 8, 44, 45, 46 and 47 who will have the temerity to say that the wild and woolly West is not "up-to-snuff" in the higher ideals in education? Yours truly, W. P. H.

Portland, Ore., April 11, 1907.

[While some of the questions may cause a smile, we think a second reading of the list will show that the questions as a whole have much to justify them. They are printed in full below. Our correspondent encloses the letter head of the company, which we also reproduce, as showing that art as well as other things making for culture is appreciated by the officers of the Pocatello Water Co. That the artist has taken some liberties will be appreciated by those who have been in Pocatello, but if Portneuf Creek is not so large and the mountains not quite so near Pocatello as the engraving seems to indicate, both creek and mountains a plenty are in evidence there.

It may not be out of place to state that Pocatello is a thriving city in southeastern Idaho, at the junction of the old Oregon Short Line and the Utah & Northern railways, now part of the Union Pacific system. Its population in 1900 was 1,046. A gravity water supply has been brought in from the mountains, and irrigation is practiced in the vicinity. The examination questions which follow were originally printed as a neat pamphlet, on linen paper.—Ed.]

POCATELLO WATER COMPANY,
Limited.

D. A. MURRAY, President. GEO. WINTER, Chief Engineer and General Manager.

Form G, Series D. February, 1896.

Questions for applicants for the position of engineer in charge of works. Applicant failing to qualify as engineer in charge of works may be available as draughtsman, transitmen, levelers or rodmen.

Some of these questions are intended to test the knowledge of the applicant, others are intended to test his wisdom. Availability for the position will be deduced from his answers on the whole.

(1) Name, date and place of birth of applicant.
(2) When and where did you commence the practice of engineering?
(3) What are your educational qualifications, (a) in English, (b) mathematics, (c) Latin, (d) French, (e) German, (f) any other languages, ancient or modern?
(4) Of what university are you a graduate?
(5) In what university or technical institution did you study engineering?
(6) Did you complete a four-year course?
(7) After completing your course in engineering did you travel abroad in Europe, Asia and Africa?
(8) What opportunities have you had of studying water-works engineering in British India, in the Valley of the Nile, in Italy, France, Germany, England?
(9) Name the great engineering works you have had opportunities of studying in the United States.
(10) Have you ever designed and carried to completion any water-works engineering of magnitude? If so, where and when?
(11) Name one or more practical ways of floating a long compound pipe and finding the actual discharge.
(12) What, in your opinion, should be the slope and cross section of a ditch through stiff, tough, clayey material ten miles long, that would discharge an equal quantity of water with a 36-in. cast-iron pipe of the same length, the head on the pipe being 150 ft.?
(13) Have you ever had any experience in sinking caissons? If so, where and when?
(14) How would you proceed to sink a shaft in a morass or in quicksand?
(15) Have you ever done any work of this kind?
(16) How would you prevent leaves from settling on a water screen?
(17) Are you familiar with the theory of the motion of a solid in a fluid?
(18) Write one or more of the conventional formulae commonly used in discussing this question and name one or more methematicians who have ably discussed it.
(19) Are you a student of the higher mathematics?
(20) Have you been examined in the differential and integral calculus? If so, where, when and with what results?
(21) Are you familiar with Professor Salmon's work on conic sections?
(22) What are your views on the question of municipal versus private ownership of water-works?
(23) Have you ever managed a private water-works plant? If so, state where.
(24) For the purpose of selling, what lines would you proceed upon to find the present value of a water plant, constructed twenty years ago, and now in good running condition?
(25) What would you consider a fair rate of interest on a water-works investment in Idaho, Montana or Utah?
(26) What considerations would guide you in deciding between a pumping plant and a gravity system?
(27) What would be the difference in elevation between three points, A, B and C, in an east and west line, respectively one mile apart, in the latitude of Montreal, an engineer's level being placed in the immediate vicinity of A and the rod readings on A, B and C being equal?
(28) Do you understand the use of the plane table?
(29) Have you ever used it, and if so, where and to what extent?
(30) Describe a proper plane table party for contouring a moderately rolling country.
(31) What experience have you had in stadia work?
(32) Within one, in what distance would you consider fairly good work?
(33) Have you ever had any experience in a trigonometrical survey? If so, where and when?*
(34) Solve, if you can, the following problem: From a point with a plane in the form of an equilateral triangle I measure 7½, 10 and 12½ miles to the respective angles. Find the side of the triangle.

*B. In the above problem it is required: (a) To construct the figure. (b) To prove the correctness of the construction. (c) To calculate the length of the side of the triangle. No judging will be considered.

(35) Have you ever handled large numbers of men engaged on public works, say 1,000 or more? If so, where and when?
(36) How would you proceed to put 1,000 or more men to work on a line of canal, for instance, on the first day of commencing work, so as to insure that every man would be promptly employed, there being no contract work?
(37) In your practice heretofore have you had any difficulties with your employers or with contractors, and if so, of what nature?
(38) State separately the cases where you have had difficulties and where everything passed off smoothly.
(39) Have you ever been superseded or discharged from any engineering work, and if so, for what reason, if you know?
(40) Are you a skilled engineering draughtsman?
(41) Can you sketch rapidly from perspective?
(42) Are you a member of the American Society of Civil Engineers?
(43) Are you a member of any other learned scientific society? If so, name it.
(44) Do you ever write for the press?
(45) In literature, who are your favorite authors?
(46) In the soliloquy of Hamlet, what is the meaning of the expression "a bare bodkin?" Put it in other words.
(47) Name six each of the greatest statesmen, scientists, soldiers, rulers, engineers, surgeons, lawyers who have lived during the last quarter of a century.

N. B.—Confine your answers to the questions. Do not attempt an essay on engineering.

OFFICE OF THE
POCATELLO WATER COMPANY
JAS. A. MURRAY, PREST.
GEO. WINTER, SUPT.
BUD OF THE TENDER HERB TO SPRING FORTH. JOB XXXVIII—27.
ENG. NEWS.

Notes and Queries.

W. B. Price, Stamford, Conn., wishes the addresses of dealers in old maps, particularly city and town maps.

RAPID WORK IN LINING FOUR LARGE STEEL CHIMNEYS WITH BRICK.

Rapid work is being done in lining with brick the four large steel stacks of the new power-house of the New York Edison Co., on the East River, at 39th to 40th Sts., New York City. These stacks have been in service, unlined, for some six months past, since the station was completed, as the station has been needed to supplement the company's first power-house, lying in the block south of the new one. A few weeks ago, however, the work of putting brick linings in the stacks was taken in hand. It was arranged that one stack at a time should be cut off from the smoke-flue and lined, each stack being cut out in turn. By a rig planned for rapid working, it has been possible to put in the brick work at the rate of about 35 feet. of height per day, so that only about 7 working days are required to line one stack.

These stacks are 22 ft. 6 ins. in diameter at the base, by 300 ft. high above the ground floor of the boiler-house. The lower section is cylindrical, while the upper 100 ft. taper slightly, the diameter at top being 21 ft. Each stack is seated on girders about 70 ft. above the floor, so that its net height is 230 ft. The steel plate shell is stiffened by several lines of vertical angle, and a circumferential channel every 20 to 23 ft. in height. The lining, which is designed to extend from bottom to top, is a single thickness of common brick laid flat in lime-mortar, all spaces between brickwork and shell being slushed full of cement mortar. The circumferential channels form seats on which the brickwork is carried in sections corresponding to the spacing of the channels.

The mason work is done from a circular platform suspended inside the stack. This platform, which clears the inside of the finished brickwork by about 6 ins., is hung from four ⅝-in. steel cables which pass over sheaves journaled in two pairs of 3 × 12-in. timbers laid across the top of the stack. These cables, coming down inside the stack and passing between platform and lining, are joined together in pairs, giving two down-hauls, each of which is attached to the drum of a hand winch fastened to one of the building columns at the ground floor. When the platform is to be raised, after the brickwork has been built up several feet, two men working at each winch can raise it 6 ft. in 5 to 6 mins. The platform is further connected to four safety-cables, also passing over sheaves in the cross-timbers at the top

of the stack. These cables contain a set of double blocks, the downhaul being made fast on the platform itself. They are used to assist in raising the platform when it is loaded heavily with bricks and mortar.

Material is handled to the working platform by a hoist passing up through a rectangular hole in the platform. The hoist, about 2 × 6 ft. in size, is hung from a cable passing over a hoisting sheave at the top of the stack, thence down along the side of the hoistway to a lead block at the ground level and horizontally to a hoisting engine. Vertically along each side of the hoistway a ⅝ in. steel cable is stretched from top of stack to ground, passing through holes in the hoist platform; these cables steady the hoist against lateral swinging.

The working force consists of ten masons and twelve laborers, the latter serving the hoist. Mortar is mixed on the ground floor ready for use, and is filled into pails (ordinary wooden water-pails) for hoisting. Bricks are hoisted in small rectangular wicker baskets, each holding about 28 bricks. The use of the pails and baskets has proved very convenient and favorable to rapid working.

The rate of progress attained in this work, as already stated, was in excess of 30 ft. in height per working day of 8 hrs. For several days a progress of 36 ft. per day was made, and the entire lining of one stack was completed in 55½ working hours. As each stack contains about 100,000 bricks, this is equivalent to about 1,500 bricks per day per man, average rate, or nearly 200 bricks per man per hour. This is by no means an exceptional rate, of course, for ordinary conditions; it is noteworthy, however, as showing that under the specialized conditions of working in the interior of a chimney it is feasible to concentrate as many men and attain as high a rate of bricklaying as in simple wall masonry. As facilitating rapid work, it must be noted that the steel shell of the stack furnished a constant guide to the mason [the bricks being merely bedded and shoved back until the mason's fingers touched the shell], so that plumbing and lining-in were largely eliminated.

Incidental to this work is the observation that the several months during which these stacks had been in constant use without lining resulted in only a very small amount of corrosion of the steel shell, even the rivet-heads being fairly clean.

The construction of these linings is being done under contract by the Murphy Construction Co., of New York City, General Builders. Mr. J. H. Ross is Superintendent for the contractor.

THE HYDRAULIC COMPRESSED AIR PLANT OF THE VICTORIA MINES, MICHIGAN.

The compression of air by the direct action of falling water was first described by Mr. J. P. Frizell, M. Am. Soc. C. E., a civil and hydraulic engineer of Boston, Mass. Later his method was improved by Mr. Charles H. Taylor, of Montreal, and it is his system that is in use to-day at Magog, Que., Peterborough, Ont., Norwich, Conn., and at the plant we are about to describe, be es one in the Cascade Range of mountains in ae State of Washington. A description of Mr. Frizell's system was given in Vol. V., pp. 345, 354 and 361, of Engineering News, and later Mr. Frizell read an article on the same subject before the Franklin Institute, which paper was published in the "Journal of the Franklin Institute," September, 1880. In May, 1901, Mr. William O. Webber, M. Am. Soc. M. E., read a paper before the American Society of Mechanical Engineers, giving the general dimensions of the Magog, Peterborough, Norwich and Washington plants, and the results of tests on the Magog plant, showing the efficiency, etc. (Eng. News, May 30, 1901, p. 406). The Norwich plant was also fully described in these columns (Eng. News, June 12, 1902, p. 470). The latest application of Mr. Frizell's method is at Rockland, Me., where a plant is being built to compress air by tidal power (Eng. News, Dec. 6, 1906, p. 585).

The hydraulic compressed air plant of the Victoria Mines is located in Ontonagon County, near Rockland, Mich. Water is taken from the Ontonagon River and after passing through the compressor is returned to the river a mile further down stream. A concrete dam 300 ft. long and 10 ft. high was built across the river 4,000 ft. up-stream from the compressor house, and a canal with a sectional area of 350 sq. ft. conducts the water from the dam to the compressors. At the foot of the canal three vertical circular and smoothly cemented shafts, each 5 ft. in diameter, were sunk to the depth of 330 ft. These shafts terminate in a chamber 57 ft. wide, 22 ft. high and 50 ft. long. The chamber then narrows to a width of 18 ft. and a height to the center of the arched roof, of 25 ft. This part of the chamber is 232 ft. long, making a total length of 282 ft. Here the chamber is reduced to a tunnel 10 ft. high, and after a run of 40 ft., it ascends on an incline to the surface.

At the bottom of the shafts are fitted steel tubes, which extend into the chamber 16 ft. These tubes flare from a diameter of 5 ft. at their connection with the shafts, to 7 ft. 4 ins. at their base, while immediately below each of them is placed a concrete spreading pier. Steel tubes are also fitted to the upper ends of the shafts and extend 6 ft. above the base of the forebay. Into these are telescoped other pipes, to which are attached the head pieces of the compressors.

By this arrangment the flow of water through the compressors is controlled, the head piece being lowered below the water level of the forebay, when the compressor is working, or raised above the water line, so that no water can pass through it, when the compressor is shut down.

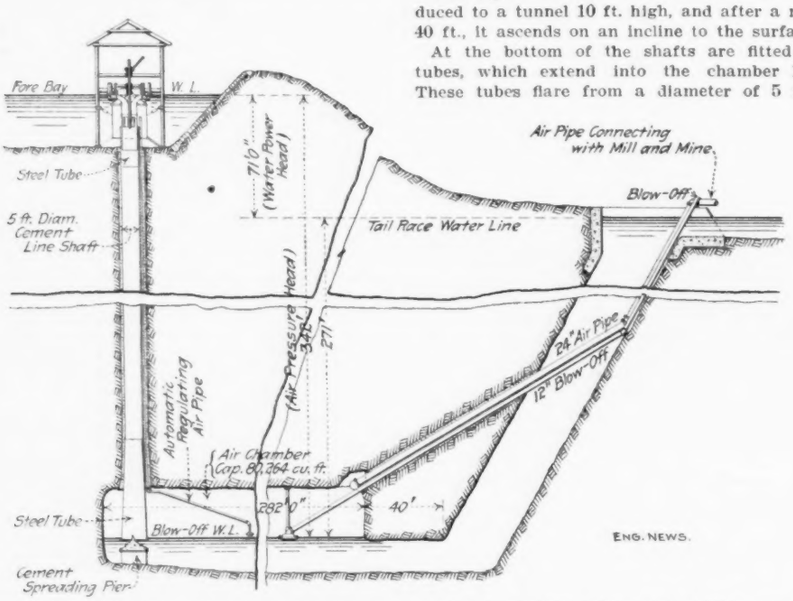


FIG. 1. VERTICAL SECTION OF TAYLOR HYDRAULIC AIR COMPRESSOR AT VICTORIA MINES, MICHIGAN.

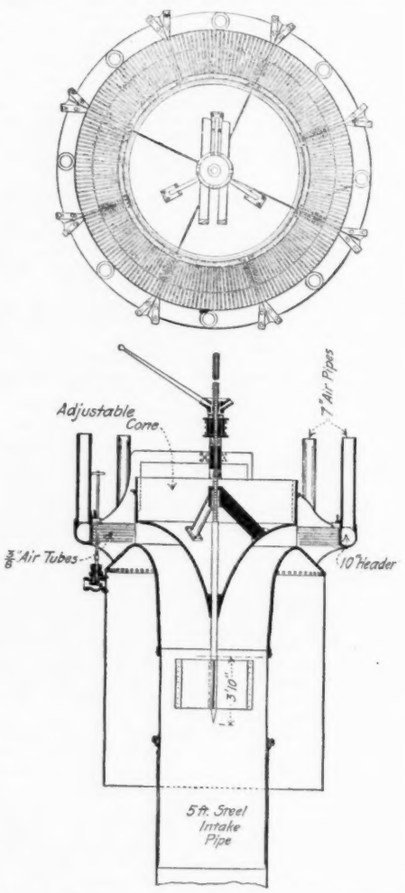


Fig. 2. Plan and Sectional View of Hydraulic Air Compressor Head Piece, Victoria Mines.

The air capacity of the underground chamber, between the water line and the roof, is 80,264 cu. ft. Out of the top of this chamber extends a 24-in. air main, while along side of it runs a 12-in. blow-off pipe. This latter pipe has its lower end on a level with the water line of the chamber, which is at such a height as to keep the down-take tubes of the compressors always sealed with water. The office of this blow-off pipe is to prevent the air pressure becoming so great as to force the water away from the down-take tubes, thereby breaking the water seal and allowing the air to escape through them. When the air pressure becomes sufficient to force the water to the level of this blow-off line, air and water escape from the blow-off pipe until the pressure is relieved and the end of the blow-off pipe becomes again sealed by the water. The air and the blow-off pipes are both cemented air-tight into a 30° tunnel, which carries them to the surface, where the blow-off terminates and the air main is continued to the mines. It may be remarked that when the blow-off is in operation it throws a mixed stream of air and water some 500 ft. into the air.

The headpiece referred to above consists of an annular pipe or header attached to the adjustable telescoping pipe. This header, which is about 10 ins. in diameter, is connected with eight 7-in. vertical intake air pipes, the upper ends of which always project above the water level of the forebay. From the inside diameter of the circular header project radially, 1,800 tubes, ⅜-in. in diameter. The top of the adjustable steel pipe is a convex casting with an adjustable concave cylindrical casting fitted therein, thus making possible the closing

of the small air tubes and thereby adjusting the air supply. The whole headpiece is supported by I-beams of the compressor house. The headpiece and concave adjusting cone can be raised and lowered by lifting screw and capstan nuts. The convex casting of the adjustable head is riveted to a larger diameter tube, thereby making each headpiece an inverted tank, or float.

The compressor is set in operation by lowering the headpiece by turning the capstan nut until the lower rim of the convex casting settles a few inches below the surface of the water in the forebay. In this position the water rushes over the ⅝-in. air tubes and passes down between the conoid castings into the vertical shaft. On passing the ends of the small air tubes the water drops away at an increased velocity, thereby creating a partial vacuum, which causes air from above to rush in. This air is taken up by the water, in the form of small bubbles, and carried down the shaft, being gradually compressed thereby. On reaching the bottom of the shaft the mixed volume of water and air is spread out in all directions by the conical cement pier. The water then slowly flows along the air chamber, while the air rises through it and accumulates in the dome of the chamber. On reaching the end of the chamber the water is free of air and flows up the inclined shaft and discharges into the tail race.

Automatic regulation of the flow of water over the small air tubes is obtained by means of a small air pipe, which runs from the air chamber up to and connects with the adjustable headpiece. The end of this pipe in the chamber is placed at such a height that, when the air pressure reaches the desired point, the pressure will raise the adjustable head and stop the flow of water over the air tubes. As soon as the air pressure in the chamber is relieved, the headpiece is lowered and the compressor placed in operation again.

As noted above, three vertical shafts are used instead of one larger one. This arrangement was decided on for the reason that a higher efficiency could be obtained during a dry season, with less water, and also that thereby a very high efficiency could be secured through a range of 1,000 to 5,000 HP. According to tests which were made by Messrs. Sperr and Hood, of the Michigan School of Mines, the efficiency of the plant, when running at about maximum capacity, is 82%. The total distance between water level of the forebay and the water level of the air chamber is 342 ft., while the distance between forebay water level and tail race level is 71 ft. This gives an air pressure head of 271 ft., which produces a pressure of 117 lbs. per sq. in. in the air chamber. The plant was designed and built by Mr. C. H. Taylor, of Montreal, and cost less than $22 per HP., while the dam and canal cost about the same. The cost of operating the plant averages about $2.25 per HP. per year, allowing 5% interest on first cost. For information regarding the plant we are indebted to an article in "The Engineering and Mining Journal" by Mr. Dwight E. Woodbridge.

A DECISION ON STREET LIGHTING BY ARC LAMPS AND THE COMMERCIAL CANDLE-POWER RATING.

The findings of the Board of Arbitration in the case of the City of Colorado Springs vs. the Pikes Peak Hydro-Electric Co., are interesting to all connected with municipal lighting service. The Board consisted of Prof. L. G. Carpenter, of Fort Collins, Colo.; Mr. E. L. Elliott, and Mr. Henry Floy, of New York City. The city claimed that the enclosed arc lamps furnished were not in accordance with the provisions of the company's franchise, which specified arcs of standard 2,000 candle-power, and that therefore a reduction should be made in all bills rendered by the company to the city, whether paid or unpaid. The Board separated the claims of both parties into several heads and made findings on each as follows:

(I.) Does the phrase "Arc lamps of standard 2,000 candle-power each" mean an arc lamp giving 2,000 actual candle-power, or if not, what was the generally accepted meaning at that time?

The experts on both sides unanimously agree that at the date of the granting of the franchise, namely, Sept. 8, 1898, there was no arc lamp in use for street-lighting purposes of 2,000 actual candle-power and therefore the phrase cannot be taken literally. The testimony of all the experts, however, shows that there was a direct, constant-current series, open arc lamp taking 9.6 amperes and consuming 450 watts at the arc which was generally accepted at that time as complying with the meaning of the phrase.

(II.) Do the arc lamps which the company has furnished, when operated under normal conditions, come within the meaning of the phrase "Arc lamps of standard 2,000 candle-power?"

There was no difficulty in reaching a conclusion on question II., since the uncontradicted testimony of the expert witnesses was to the effect that the lamps furnished by the company did not come within the meaning of this phrase.

(III.) If the lamps furnished by the company, when operated under normal conditions, have not fulfilled the requirements of the phrase "Arc lamps of standard 2,000 candle-power," what is the extent of the overcharge measured in dollars and cents due to such deficiency?

In arriving at an answer many difficulties were encountered arising from the present imperfect state of the commercial photometry of arc lamps, the meagerness of evidence on some points, the variance of expert testimony on other points, the difficulty of selecting a satisfactory standard of comparison, and of estimating, on a dollar and cents basis, the difference in arc lamps of such widely varying characteristics. The experts generally agree that candle-power measurements alone do not fully express the effective values of arc lamps for street lighting, and that while the lamps used fell short of meeting the requirements of an "arc lamp of standard 2,000 candle-power," they had some compensating advantages. After carefully weighing such evidence as was presented, the arbitrators reached the conclusion, everything considered, that 20%, in the present case, is a fair estimate in dollars and cents of the deficiency of the lamps furnished by the company as compared with "arc lamps of standard 2,000 candle-power," both types considered as operating under normal conditions.

(IV.) Was the service which the company actually furnished from the lamps in use such as might reasonably be expected, and if not, what was the overcharge expressed in dollars and cents due to defective service?

The company admitted that during a portion of the period involved in this controversy, its lamps were not operated under normal conditions, and placed in evidence a complete set of station records covering the entire period. From these records it appears that during a portion of the time the lamps were operated below their proper wattage, and for another portion of the time above their normal wattage. There appeared to be a discrepancy, however, between the wattage delivered at the lamp terminals, as computed from the station readings, and the wattage as actually measured by tests at the lamps.

The discrepancy was 6¼%, which has been used as a correction constant. The corrected wattage thus obtained was used to determine the candle-power from the data given in Prof. Matthews' tests, made on lamps taken from the city circuits. These candle-powers compared with the candle-power of a lamp operated at its normal wattage (430) were expressed in per cent., which gave the means of determining the deficiency or excess of service supplied.

(V.) Was the city estopped from claiming any refund?

The arbitrators requested briefs upon question V. from counsel on both sides, and the conclusion was reached that there was no intention on the part of either party to evade the obligations of the contract. The good intention of the company could not, however, be held as justification for an actual deficiency in service, which has been shown to exist. On the other hand, the fact cannot be overlooked that the city should have been advised in regard to a matter of such importance, before accepting the service furnished by the company, or at least before having allowed it to continue for so long a time without protest. For these reasons the arbitrators consider that the responsibility is divided between the company and the city for the period to June 1, 1906, and therefore conclude that the reduction on account of overcharge for this period should be figured on a basis of 10% in place of the 20% taken after June 1, 1906.

THE DEVELOPMENT OF MONTREAL HARBOR is to have the continued aid of the Dominion Government. A resolution passed by the House of Commons authorizes the loan of $3,000,000 to the Montreal Harbor Commissioners at 3% interest. In the past ten years the government has loaned over $7,000,000 for improvement of the port of Montreal in the endeavor to build it up to be the principal harbor of the country.

RAIL FRACTURES ON RAILWAYS IN NEW YORK STATE.

Definite figures concerning rail fractures, which seem to be the most extensive ever published, are given by the New York State Railroad Commission in a bulletin issued a few days ago. They are contained in tables showing the total number of rail breaks that occurred on the principal steam railways in New York State during the winter months, January to March, inclusive, of the three last years; and showing how they are distributed as to size of rail and date of rolling. The totals show not only a surprisingly large number of fractures, but also a striking increase during the year just past, as exhibited in the following summary which we have compiled from the Commission's figures:

SUMMARY OF RAIL FRACTURES ON PRINCIPAL STEAM RAILWAYS OF NEW YORK STATE FOR JAN., FEB. AND MARCH, 1905-1906-1907.

Name of Railway.	*Miles main track in New York.	1905 Number of fractures.	1905 Number per mile of main track.	1906 Number of fractures.	1906 Number per mile of main track.	1907 Number of fractures.	1907 Number per mile of main track.
Del. & Hud..	857	60	.07	60	.07	93	.11
Lehigh Vall..	807	125	.15	50	.06	103	.13
Rutland	171	10	.06	7	.04	20	.12
N.Y., N.H.&H.	160	10	.06	2	.01	29	1.8
B. & M.....	146	34	.23	7	.05	5	.03
B. R. & P....	202	27	.13	4	.02	256	1.32
N. Y., O.&W.	526	No record		224	.05	10	.02
Penna.	332	140	.42	186	.56	205	.62
Lake Shore..	158	227	1.44	34	.22	505	3.24
N.Y. C & H.R.	4,183	469	.16	288	.07	1,244	.30
Erie 1,301		76†	.06	142†	.11	429†	.33
D., L. & W..	630	153	.24	22	.03	115	.18
	9,474	1,331 or 0.14 per mi.	...	826 or 0.09 per mi.	...	3,014 or 0.33 per mi.	...

*Mileage from 1905 Report of State Railroad Commission.
†Incomplete.

The total breakages were 1,331 in the first three months of 1905, 826 in the same period of 1906, and no less than 3,014 in the first three months of the current year. More expressive, perhaps, are the figures when reduced to breakages per mile of main track [including all trackage except sidings, yards and turnouts, and counting two for each mile of double-track line, etc.]. These show that in the three winter months of 1905 there was on the average one breakage for every seven miles of the 9,474 miles of main track included in the table; but in 1907 the frequency had more than doubled, one for each three miles. The alarming meaning of this latter figure may be thus pictured: Taking the main line of the New York Central between New York and Albany, 140 miles, part three-track and part two-track, and transferring the average 1907 figure to this stretch of track, we would have during the 90-day period in question slightly over one fracture every 24 hours!

The totals are classified by the Commission in two tables reproduced here in condensed form as Tables I. and II. The first classifies the fractures according to size of rail, expressed by its weight in pounds per yard. Even cursory inspection of this table shows that the great number of the breaks occurred in the heaviest rail commonly used: in 1907 the 100-lb. rail, in 1905 the 80-lb. rail. This fact may be explained by two conditions, first, that the lighter rails, laid in earlier years, have had the bad, brittle ones among them fairly well weeded out by breakage before the period covered by the table; second, that the heaviest sizes of rail are most seriously affected by segregation, with its resultants of non-uniform quality, local hardness, brittleness, etc., by the practice of finishing at too high a heat, and by pipe and seam formation. Which of these two conditions is the chief underlying factor is not determinable with certainty from these figures. But the great increase for 1907, with its coincident high breakage of 100-lb. rail, suggests that this heavy size is very much more liable to bad quality resulting in breaks than the 80-lb. rail of the preceding years.

Table II. shows clearly that breaks are far more frequent in the first year or two of a rail's service than in subsequent years. In fact, summing the totals for the three 3-month periods, we find that 2,841 breaks occurred in the first three years of use; that is, more rails broke be-

fore reaching the age of 3 yrs. than broke at all later periods. The enormous number of breaks in the first year is particularly noteworthy. Attention is also to be directed to the fact that of the total breaks of 1907 that are listed as rolled in 1906, four breaks occurred in rails rolled in 1907, that is after perhaps 60 days of service.

A good side-light on the relation of age to frequency of fracture is obtained by the following tabulation, made up from Table II. by summing the figures for 1905, 1906 and 1907 and dividing by 3 to get the average:

AVERAGE RAIL BREAKS PER WINTER DURING 1905-1906-1907.

Age of rail at time of fracture.	No. of breaks in 3 winter months, Jan.-March.	No. of breaks in 3 winter months for each year of origin.
1	418	418
2	305	305
3	224	224
4	87	87
5	104	104
5 to 10 yrs.	237	47
10 to 15 yrs.	122	24
15 to 20 yrs.	145	29
Older.	81	..

The last column of this table may be plotted as ordinates with years of age as abscissas, yielding a curve whose sharp hyperbolic form is noteworthy. That is, very roughly, if 500 rails break in one year at an age less than 1 yr., in the same period only about 50 rails of an age of 10 yrs. will develop fractures.

The figures here given are very strong and direct confirmation of the claims made in recent years by those engineers who have been fighting for increased stringency of specification requirements for steel rails. This fight, much of it carried on in the American Society for Testing Materials, has aimed chiefly at (1) specifying a greater amount of cropping from the upper end of the ingot, and (2) making a somewhat more severe drop test than hitherto accepted as standard. Both requirements are intended to combat brittleness and internal weakness of structure. The progress of events as indicated by the remarkable increase in rail breaks shown in the present figures will doubtless further these efforts.

RAPID TRANSIT SUBWAY EXTENSION in New York City has received a setback by the failure of the Board of Rapid Transit Railroad Commissioners to secure any bids for two new subway lines in response to a recent invitation for bids. A map and brief description of the various routes for new subways projected by the Board as desirable transit lines were given in our issue of Dec. 27, 1906, pp. 681 and 693. Of the seven routes there shown, two, namely, the Seventh and Eighth Ave. route and the Lexington Ave. route, were accepted by the city authorities and advanced to invitation for bids. These two routes are north-south routes in the Borough of Manhattan. The first, beginning at the Battery, the southern point of the island, extends up the district west of Central Park, and terminates at 154th St. and Eighth Ave., all in the Borough of Manhattan. The Lexington Ave. route, also beginning at Battery Park, extends up the east side of the island, crosses the Harlem River in the neighborhood of 130th St. into the Borough of the Bronx, and there fans out into three terminal branches. Both routes touch the present rapid-transit subway at a point where connection may be made with it, the western route at Broadway and 47th St., the Lexington Ave. route at 42d St. by a short connecting spur from Lexington Ave. to Fourth Ave. As already noted, no bids whatever were received by the day fixed therefor. The Interborough Rapid Transit Co., operating the existing subway, suggested informally that it would be willing to construct and operate part of both routes, namely, the Lexington Ave. route above 42d St. and the western route below 42d St. This arrangement would enable the company to form the system into two complete lines, an east and a west side line, in place of the three lines which would be secured by the proposed two new lines. But the Interborough company asserts that it can afford to make its offer only if the form of contract proposed by the Board of Rapid Transit Railroad Commissioners be made much less favorable to the public and the city. In the meantime, the existing subway and elevated railways are congested to suffocation in the rush hours, and the Interborough company, which also operates the elevated lines, is asking a perpetual franchise for additional tracks on one of the East Side elevated lines to enable it to run an express service on this line.

BIDS FOR SUBMARINE BOATS have been received by the Navy Department of the United States. There is an appropriation of $3,000,000 for such vessels. Three bids were received: from the Lake Torpedo Boat Co. for boats of 235 to 500 tons (15 knots), at $198,000 to $233,000 each, if more than five are contracted for; from the Subsurface Boat Co., for 250-ton vessels (19 knots), at $250,000 each; from the Electric Boat Co., for boats of 274 or 340 tons (13 knots), at $312,000 or $379,000 each. At the same time a Trial Board of the Department is conducting tests of submarines of the Holland and the Lake type in Narragansett Bay.

SWEDEN IS BUYING UP ITS WATER POWERS by government appropriations, in order to secure sources of power distributed over the country to furnish energy for operating the railway system. The government has already initiated a large-scale trial of high-voltage single-phase electric traction for main-line operation, and in furtherance of the same views as led to this experiment it is now surveying and acquiring the various water powers suitable for developing large blocks of power. Last year the national assembly appropriated about one and a half million dollars for this purpose, and nine-tenths of this sum has already been spent. The "Schweizerische Bauzeitung" reports that the falls of Motala, Svarta, Karse and Hammarby have been bought. The great Trollhätta falls and the falls of Elfkarleby had previously been acquired.

A HEAD-END COLLISION occurred on the New York, Philadelphia and Norfolk R. R. on April 25 at Salisbury, Md., quite recently the scene of another disastrous wreck which is still fresh in the public mind. A southbound local passenger train and a northbound freight came together, killing instantly the engineer of the passenger train and seriously injuring two other trainmen.

A BROKEN BRAKE CHAIN on a trolley car caused a bad accident on one of the steepest hills in Newark, N. J., on April 27. On account of the extreme grade at this point the company uses its smallest cars which are equipped with the ordinary ratcheted chain-drive brake. The car had almost reached the summit of the hill when the trolley pole slipped and the car started to go back. As the pressure was put on the brake the chain snapped and the car got away. At the bottom of the hill is a sharp curve and as the car, heavily laden with 60 passengers, reached the curve, it jumped the tracks and was finally stopped against a tree. Twelve persons were more or less injured.

AN EXPLOSION on April 26, in the Morgan slope mine at Black Diamond, Wash., caused the death of four men and the fatal injury of five more; fourteen others were injured more or less severely.

THE ANGLEUR COAL MINE at Liege, Belgium, was flooded on April 29, and 18 men were entombed; 200 miners were saved by means of quick work with the hoisting apparatus. On April 30 nine bodies were recovered of those who were imprisoned. The flood burst into a gallery more than 200 ft. underground.

TO EXTINGUISH THE FIRE which for four weeks had been burning in the Homestake mine at Lead, Colo., the waters of Whitewood Creek were turned into the workings at the 300-ft. level on April 22. It will probably take thirty days to flood the mine to that level, and twice that length of time to clear the water from the workings afterwards. A complete shut-down in all departments took place on April 22, to continue for thirty days. Up to that time the fire had already cost the company over $1,000,000, and before the mine is again in working order it is estimated that the loss will be increased by as much more.

AT THE FOUSTWELL MINE, of the Berwind-White Coal Co., near Johnstown, Pa., the seven men who had been imprisoned since the afternoon of April 26 in heading No. 4, by the flooding of the workings, were rescued at 10 o'clock on the night of April 30. The flood was caused by the blowing out of a bulkhead between the mine and an abandoned working, and the men would have been drowned had they not been able to retreat to the high point from which they were afterwards taken. By persistent tapping on the air pipes the men successfully conveyed to the workers at the surface the in-

TABLE I.—BROKEN RAILS REMOVED FROM THE TRACKS OF THE PRINCIPAL STEAM RAILROADS IN THE STATE OF NEW YORK. DURING JANUARY, FEBRUARY AND MARCH, 1905—1906—1907. CLASSIFIED BY WEIGHT OF RAIL IN POUNDS PER YARD.

Name of Railway.	Total miles Main Track in New York.	January, February and March, 1905								January, February and March, 1906								January, February and March, 1907							
		50-59	60-69	70-79	80	85	90	100	Total	50-59	60-69	70-79	80	85	90	95	100	Total	50-59	60-69	70-79	80	85	90	95	100	Total
Del. & Hudson	857	.....	.....	.....	60	....	....	....	60	.....	.....	.....	60	....	....	....	....	60	.....	.....	.....	.....	....	93	....	....	93
Lehigh Valley	807	9	11	2	76	....	27	....	125	7	3	2	2	....	10	....	....	50	15	2	3	45	....	38	....	....	103
Rutland	171	.....	.....	2	8	....	....	....	10	.....	.....	3	4	....	....	....	....	7	11	2	1	6	....	....	....	....	20
N.Y. N.H. & Harlem	160	.....	.....	10	....	....	....	....	10	.....	.....	1	....	....	....	....	1	2	.....	.....	28	....	....	....	....	1	29
Boston & Maine	146	.....	6	25	....	3	....	....	34	.....	.....	2	....	5	....	....	....	7	.....	.....	2	....	3	....	....	....	5
Buff. Roch. & Pitts.	203	.....	.....	.....	26	....	1	....	27	.....	.....	.....	4	....	....	....	....	4	.....	.....	.....	13	....	243	....	....	256
N. Y. O. & W.	526	.....	.....	No record	....	....	....	....	.....	4	3	13	....	....	....	4	....	24	2	....	4	....	....	4	....	....	10
Pennsylvania	332	16	119	3	1	1	....	....	140	4	150	.....	13	19	....	....	....	186	10	111	4	8	72	....	....	....	205
Lake Shore	158	.....	5	.....	222	....	....	....	227	.....	2	.....	16	....	....	....	16	34	.....	4	.....	6	....	....	....	495	505
N. Y. C. & H. R. (incl. West Shore)	4183	.....	34	17	219	....	....	199	469	.....	24	1	183	....	....	....	80	288	.....	47	24	374	....	....	....	799	1244
Erie	1301	.....	.....	10	30	....	36	....	*76	.....	45	.....	7	....	90	....	....	142*	.....	84	8	38	....	299	....	....	*429
D. L. & W.	630	.....	45	31	77	....	....	....	153	2	4	7	9	....	....	....	....	22	.....	.....	6	105	....	4	....	....	115
Total	9474	25	220	100	719	4	64	199	1331	17	231	29	324	24	100	4	97	826	38	250	80	595	75	677	4	1295	3014

* Record incomplete.

TABLE II.—BROKEN RAILS OF TABLE I. CLASSIFIED ACCORDING TO DATE OF MANUFACTURE.

	January, February and March, 1905 — Year Rail was Rolled										January, February and March, 1906 — Year Rail was Rolled										January, February and March, 1907 — Year Rail was Rolled									
	1904	1903	1902	1901	1900	95-99	90-94	85	89	Before 1885	Total.	1905	1904	1903	1902	1901	96-00	91-95	86	90	Before 1886	Total.	1906 and 1907	1905	1904	1903	1902	1897 to 1901	1892 to 1896	1887 to 1891	Before 1887	Total.
Del. & Hud.	54	6	....	....	....	....	....	....	....	60	....	....	56	4	....	....	....	....	....	....	....	60	93	....	....	....	....	....	....	....	....	93
Lehigh Vall.	1	3	2	4	16	....	1	99		125	1	....	3	2	4	....	....	40	....	....	....	50	1	2	7	3	10	6	....	74	....	103
Rutland	....	....	....	2	2	1	3	2	....	10	....	....	....	....	2	2	1	2	....	....	....	7	....	....	....	....	....	6	....	1	13	20
N.Y.,N.H.&H.	....	....	....	....	....	....	7	3	....	10	....	1	....	....	....	....	1	....	....	....	....	2	3	23	....	....	....	....	2	1	....	29
Bos. & Maine	....	2	....	1	2	6	17	6	34	....	....	....	1	2	3	1	....	....	....	....	....	7	....	1	....	....	....	4	....	....	....	5
Buff.,Rh.&Ph.	1	2	2	....	2	19	1	....	....	27	1	....	....	....	....	3	....	....	....	....	....	4	239	8	1	....	....	3	5	....	....	256
N. Y. O. & W.					No Record.					....	4	....	6	....	4	3	6	1	24	....	....	1	2	....	3	2	....	2	10			
Pennsylvania	....	....	1	....	22	73	25	3	16	140	3	5	....	11	1	86	17	59	4	186	18	11	19	6	11	107	15	7	11	205		
Lake Shore	105	8	....	16	33	60	5	....	....	227	16	1	3	....	4	7	3	....	....	34	176	319	....	....	....	6	4	....	....	505		
N.Y.C.& H.R.	208	29	29	58	62	81	35	7	10	469	13	77	30	53	22	56	19	13	5	288	155	240	495	60	67	142	27	35	23	1244		
Erie	....	4	4	....	16	10	29	10	....	76*	3	20	18	19	3	27	3	4	45	142*	156	87	26	4	3	24	24	25	80	429*		
D. L. & W.	2	1	17	1	2	18	68	23	21	153	....	1	2	3	1	....	8	1	6	22	4	4	6	9	25	9	54	3	1	115		
	371	56	57	81	156	214	179	164	53	1331	37	165	60	95	39	188	55	126	61	826	845	695	555	84	116	310	133	146	130	3014		

* Record incomplete.

formation that there were seven men, and that they were located in heading No. 4—the taps being regulated in sevens and fours, and correctly interpreted by the rescuers. When the water level had been reduced, by means of the regular pumps and extra ones installed for the emergency, so that there was head-room between the roof of the tunnel and the water surface, four men waded through the water nearly to the imprisoned men, but were forced to wait until the pumps had operated about two hours longer. Then another and more successful attempt was made, and the men, greatly weakened through loss of sleep and nourishment, were supported through the gallery to the surface.

A CLOUDBURST over Capital Hill, a suburb of Oklahoma City, flooded that locality on the night of April 28, and seven persons were drowned. The flooded area covered several square miles, and much damage was caused to farm and railway property—a mile of track on the Santa Fé being washed out and traffic brought to a standstill.

A NEW PIER AND PIER-SHED COLLAPSED at Locust Point, Baltimore, Md., about noon of April 27, killing probably five men of the working force engaged in completing the pier-shed. The pier was being built for the Baltimore & Ohio R. R. Co. for docking ocean steamers. It is about 950 ft. long by 160 ft. wide, with full-length water frontage on one side, while on the other side the shore line is at about the middle of the length. The pier was founded on piles, with a retaining wall around the outside to hold earth filling. The deck was built of reinforced concrete. A two-story steel pier-shed designed for heavy loading was erected on the pier; this was about complete as to framework, and finishing work, such as putting on corrugated steel sheathing, installing elevators and the like was going on at the time of the accident. For some time before the collapse settlement of the outer portion of the pier had been observed. It was realized that the foundation piling was showing signs of insufficiency, which was attributed in some quarters to washing away of the mud on the bottom of the Patapsco River. Work was in progress for strengthening the foundations. More piles were being driven, and sand was being dumped around the outside to replace the mud. The pier seemed to have moved slightly outward, the outer half cracking away from the inner half. About 11 a. m. of April 27 the outward settlement increased rather rapidly, and a few minutes later that part of the pier lying wholly outside of the shore line suddenly sank, by an amount varying irregularly up to some 20 ft. The superstructure was extensively wrenched and broken and much of the shed collapsed. A brick fire-wall between the inner and outer half of the structure also fell. About 100 men were at work on the pier at the time; many of them were injured, three were killed and a number are reported missing.

The foundation conditions at the site are apparently not favorable to heavy loading. Available information is not entirely in agreement as to the subsoil. A published statement credited to B. & O. officials states that after a deep layer of mud, a 5-ft. bed of iron-ore, and sand and clay, rock was found at 55 ft. below water level; that this was assumed as the foundation stratum and all piles were driven to this depth; that the loading per pile was about 9 tons, although the ultimate or failure load should be at least 20 tons per pile. The view is advanced that scour of the mud decreased the lateral support of the piles and allowed them to bend outward. Other information indicates that the hard stratum relied on to hold the piles overlay strata of soft material, into which the points of the piles probably penetrated. As the piles were driven very close together the vertical load on the rectangle of hard stratum may have exceeded its shearing strength, dropping it down into the muck below. The information on hand is, however, quite insufficient to allow more than the statement that the pile foundation proved inadequate for the load imposed by the pier, filling and pier-shed, that settlement had appeared and that efforts were being made to strengthen the foundation. The structure cost about $750,000, and the loss will probably be at least half this amount. The McLean Contracting Co. was the general contractor, and the Baltimore Bridge Co. was contractor for the fabrication and erection of the superstructure of the pier.

A TORNADO almost totally destroyed the village of Hemming, in Cook County, Texas, on April 27, and caused eight known deaths. A number of houses were demolished at Valley View; and at Celeste a big cotton mill was blown down, though no one was injured; at Sulphur Springs trains were stalled by hail that banked in drifts 2 ft. deep on the tracks.

THE ACCIDENT IN THE 38TH ST. STATION of the New York Edison Co., on April 20, described in our last issue, p. 468, appears to have been of somewhat different nature from what the earlier reports indicated. According to present information the turbine unit did not run away and go to pieces by centrifugal force. What happened apparently was a short circuit and consequent burning out of one of the main feeders supplied by this station. The enormous momentary load due to this short circuit caused the almost simultaneous breakdown of three large units, one turbine-driven and two engine-driven. The alternator of the turbine unit appears to have burned up, and the engines of the other units were badly wrecked.

PACIFIC COAST RAILWAYS IN MEXICO are an important feature in the development of the railway system of that country. The Mexican Central Ry. extensions to Manzanillo and Acapulco were described in our issue of April 4. The Southern Pacific Ry. is now building a line from the port of Guaymas, the present terminal of the Sonora branch, south to the port of Mazatlan, and thence to Guadalajara, where connection will be made with the lines of the Mexican Central Ry. The new line will be nearly 800 miles long, and will lie along the Pacific coast.

AN ELECTRICALLY OPERATED PAPER MILL to be installed at Hokaido, on the Pacific coast of Japan, is the result of investigations of representatives of the Oji Paper Co., who have been in this country for some time past. Orders for machinery have been placed, amounting to over a million dollars. A hydro-electric plant with a 15-mile transmission line will drive the machinery of the new mills. Current will be delivered from General Electric Alternators at 3,450 volts to twelve step-up transformers. The transmission will be at 45,000 volts and the driving motors for the pulp machinery will operate on 2,000 volts.

WHOLESALE PRICES IN THE UNITED STATES from 1890 to 1906 have been reported on at length by the U. S. Department of Commerce and Labor in its bulletin for March. In its summary the bulletin states that, considering the 258 commodities reported on as a whole, wholesale prices during 1906 reached a higher level than at any other time during the 17 years covered by the study. The prices for 1906 averaged 5.6% higher than those for 1905 and 36.5% higher than those for 1897, which was the year of lowest prices during the 17 years covered. It is also noted that the average prices for 1906 were 22.4% higher than the average for the 10 years from 1890 to 1899. Of the 258 articles covered, 30 showed no change in average price during 1906, 50 showed a decrease as compared with the previous years, and 178 showed an increase. The following classified summary shows the per cent. of increase or decrease in average prices for 1906, as compared with those for 1905, and also the number of articles in each of the nine classes given that increased or decreased in the time:

Group.	Number of commodities.	Per cent. of increase.	Per cent. of decrease.	Number of commodities showing. Increase.	No change in price.	Decrease.
Farm products...	16	...	0.5	10	...	6
Food, etc.....	53	3.6	...	28	5	20
Cloths & clothing.	75	7.1	...	66	5	4
Fuel & lighting..	13	0.5	...	7	1	5
Metals & impl'm'ts	38	10.4	...	29	7	2
Lumber & building materials...	27	9.6	...	24	...	3
Drugs & chemicals	9	...	7.2	3	1	5
House furn. goods	14	1.7	...	4	9	1
Miscellaneous	13	7.4	...	7	2	4
All commodities..	258	5.6	...	178	30	50

PERSONALS.

Prof. F. Y. Wren, Walker Professor of Mathematics at Tufts College, has been appointed Dean of the Faculty of Arts and Sciences, to succeed Dean Shipman.

Mr. Albert W. Newton has been appointed General Inspector of Permanent Way and Structures of the Chicago, Burlington & Quincy R. R., with headquarters at Chicago.

Mr. Harry C. Ferris, son of the Vice President of the Toledo & Ohio Central R. R., and formerly Division Superintendent with that road, has been appointed General Manager of the United Railroads of Mexico.

Mr. A. Swartz, formerly Chief Draftsman of the Construction Department of the Erie R. R., at the New York offices, has been appointed Division Engineer for that road, with headquarters at Huntington, Ind.

Mr. Franklin P. Fish, M. Am. Inst. E. E., who has been president of the American Telephone and Telegraph Co. for a number of years, has resigned. Mr. Theodore N. Vail has been elected as his successor.

Dr. Alexander N. Winchell, of the Montana State School of Mines at Butte, has been elected to the chair of mineralogy and petrography of the University of Wisconsin, formerly held by Prof. William H. Hobbs.

Mr. Frank J. Schnauber, at a meeting of the Syracuse Park Commission held April 24, was engaged as engineer to superintend the engineering work in connection with the new park system. Mr. Schnauber was formerly City Engineer of Syracuse.

Mr. Augustus C. Hepp has been appointed to a position on the engineering staff of the American Dittler Sewage Disposal Co., of New York. Mr. Hepp has been connected with the engineering departments of Brooklyn and other cities, and with the Long Island R. R.

Mr. J. R. Hopson, formerly Superintendent of the Berkshire-Naugatuck division of the New York, New Haven & Hartford R. R., has been advanced to the Superintendency of the New York division, to succeed Mr. A. R. Whaley; the latter has resigned to enter the service of the New York Central.

Prof. Frank Soulé, Dean of the College of Civil Engineering of the University of California, will make a tour of Europe during the coming college year, having been granted a leave of absence for that purpose. Prof. Soulé has been in the service of the university for 38 years.

Mr. John C. Wait, M. Am. Soc. C. E., Attorney and Counsellor-at-Law, 220 Broadway, New York City, and Mr. George H. D. Foster have formed a partnership for the general practice of law under the firm name of Wait & Foster, Attorneys and Counsellors-at-Law, with offices at 38 Park Row, New York City.

Mr. Ira A. McCormack, General Superintendent of the electric zone of the New York Central R. R., is to be promoted to the position of Assistant to the General Manager, Mr. A. H. Smith. Mr. McCormack, it is reported, will spend a considerable time in Europe studying the signal systems and safety devices employed by the railways of England and the continent.

Mr. J. V. N. Dorr, Metallurgical Engineer, has opened an office in Denver. Mr. Dorr is one of the owners of the Lundberg, Dorr & Wilson properties in Terry, S. D. He has just completed the remodeling of the 300-ton Kildonan mill for the Mogul Mining Co., and is the inventor of the Dorr classifier which is coming into extensive use in wet crushing cyanide mills.

Mr. W. H. Maddocks has been appointed Assistant Superintendent of machinery and equipment for the Missouri, Kansas & Texas R. R. at Parsons, Kansas; Mr. William O'Herin, the Superintendent, who was severely injured several months ago while in charge of the work at a wreck near Deerfield, Mo., has been granted an indefinite leave of absence and has gone to Bermuda in the hope of regaining his health.

Mr. Carroll B. Shipman, former Erecting Engineer for the Westinghouse Electric and Manufacturing Co., has resigned that position to open an office as consulting engineer at 604 Mission St., San Francisco. He was in charge of erection work at the Shawinigan Water & Power Co.'s plant, the Montreal Light, Heat & Power Co.'s plant, and the plant of the Helena Power Transmission Co., at Butte.

Mr. R. A. Gray has been appointed Division Engineer of the Valley Division of the St. Louis, Iron Mountain & Southern Ry., with office at Monroe, La. He succeeds Mr. S. Q. Gibson, resigned. Mr. F. Wolffe has been appointed Assistant Engineer, with headquarters at the same place. Mr. G. A. Jameson has been appointed Assistant Division Engineer, with headquarters at McGehee, Ark.

Mr. Harry C. Heermans, formerly of the firm of Heermans & Lawrence, which company had operated the Corning (N. Y.) Water Works for the past 30 years, has been made Manager of the Hoquiam Water Co., of Hoquiam, Wash., and has purchased the Olympia Water Works at Olympia, Wash., which he will also manage. The firm of Heermans & Lawrence was dissolved some time since, owing to the death of Mr. Thomas Lawrence.

Mr. Charles M. Mills, M. Am. Soc. C. E., Principal Assistant Engineer in subway and elevated railway construction with the Philadelphia Rapid Transit Co., sailed for Naples on April 20 on the S.S. "Prinzessin Irene." He expects to be gone six weeks, and while abroad will investigate the elevated railways of Berlin and Elberfeld, and the subways of Paris and London. During his absence his duties will be carried on by Mr. A. L. Phillips, Assistant Engineer, under the direction of Mr. Wm. S. Twining, Chief Engineer of the company.

Capt. J. Francis Le Baron, M. Am. Soc. C. E., has completed a two years' contract with the United States & Nicaragua Co. as consulting engineer in Nicaragua, and will return to this country in May and resume his practice as Consulting Engineer at his former office, 812 Park Building, Cleveland. During his two years' residence in Nicaragua, Capt. Le Baron has been engaged in an exhaustive study of the hydro-electric powers of Eastern Nicaragua, and has laid out and designed 25 systems for operating different mines, sev-

eral of which are now in process of construction. He has also designed mills and works for three mines, with cyanide plants—one being now under construction, besides designing improvements for the navigation of the Prinzapulka River and a system of jetties for the entrance to the latter stream.

Major Cassius E. Gillette has resigned as Chief of the Bureau of Filtration of Philadelphia. His resignation was made at the request of Director of Public Works Stearns, who came into office on Jan. 1. Major Gillette was ordered to Philadelphia by Secretary of War Taft in the summer of 1905, to aid in investigating the frauds in the Philadelphia filtration contracts.

These frauds had been under investigation by Hon. Elihu Root before his appointment as Secretary of State, acting as Counsel for Mayor Weaver and the Philadelphia citizens associated to secure reform of the municipal government. The appeal went from Philadelphia to the Government at Washington for some engineer who could be trusted to go to the bottom of the Philadelphia filter frauds, and who could not be diverted from the task by either bribes or threats. Major Gillette had achieved a national reputation through his heroic work in exposing and bringing to justice O. M. Carter and his co-conspirators Greene and Gaynor; and Secretary Taft selected Major Gillette as the man for the Philadelphia work.

In October, 1905, Major Gillette, in conjunction with the late John D. MacLennan, made a report (Engineering News, Nov. 2, 1905), showing extensive frauds and collusion in the conduct of the filter construction works and computing a loss to the city of nearly six million dollars through these frauds. The powerful political interests which were hit by this report then endeavored to have Gillette recalled from his work in Philadelphia to duty in the War Department and when the executive departments turned them down they went to the Military Committee of the House and succeeded in securing an order terminating Major Gillette's furlough and ordering him to duty in army service. In this situation the supporters of honest government in Philadelphia appealed to Major Gillette to remain in the city's service, complete the exposure of the fraudulent work, and take charge of the completion of the filtration plant. A number of them joined in a bond guaranteeing the payment of his salary for a term of years, and under their urgent solicitation he resigned his commission in the Corps of Engineers.

Political opposition and changes have greatly hampered the work since Major Gillette took charge; but it is now so near completion that if no further delays occur filtered water for Frankford and Kensington should be turned on by May 15, and early in July there should be enough filtered water available to supply nearly the entire city.

Mr. Frederick C. Dunlap, M. Am. Soc. C. E., formerly engineer in charge of the Torresdale Filtration Plant but lately in the employ of the New York Water Commission, has been appointed to succeed Major Gillette.

Obituary.

William Fife, senior member of the firm of Fife & Son, Contractors and Masons, Indianapolis, Ind., died in that city April 22, of heart disease, aged 76 years. He was born at Edinburgh, Scotland, but his parents removed to the United States, while he was three months old and settled near Syracuse, N. Y. He removed to Indianapolis in 1889 and since then had done much work for the city.

Dr. J. Ohey, author of "The Analysis, Detection and Commercial Value of the Rare Metals," and a chemist and metallurgist of some note, died at Denver recently at the age of 68. During the early eighties he was assistant professor of chemistry at the University of Pennsylvania, and in recent years had devoted much attention to the study of rare minerals and metals, on which subject he was a well known authority.

David Willcox, until April 10 the President of the Delaware & Hudson R. R., committed suicide on Wednesday, April 24, by shooting himself in the head while en voyage to this country on the S. S. "Barbarossa." Ill-health is assigned as the reason for his act; he had sought health in a trip abroad. Mr. Willcox was also president and a director of the United States Traction Co. of Albany, and a member of the executive committee of the Southern Pacific Co.

Howard L. Ellender, a civil engineer, and at the time acting superintendent of the McLean Contracting Co., was killed in the collapse of a concrete pier at Baltimore on Saturday, April 27. He gave up his life that others might save themselves. Noticing that the concrete was cracking, he went out on the pier to look it over; he did not then order the men off the structure, but a few minutes later, and just preceding the crash, he ran out to warn them; on his way back he was instantly killed by a falling wall. He leaves a widow.

Thomas Whinston Peeples, who for nearly a quarter of a century was chief engineer of the New York Elevated Railroad system, died on April 27, after a short illness, at his home in East Orange, N. J. He was born in 1830 in Harrisburg, Pa. At the age of nine he went to work in the shops of the Pennsylvania R. R., located there, and at 21 was made master mechanic of the Harrisburg division of the road. When the Civil War broke out he organized a company of men among the employees of the road and went to the front with Company E. of the 201st Pennsylvania Volunteers. His services to the road had been so valuable that President Scott sought and obtained President Lincoln's consent for his return to the work; he remained in the service of the Pennsylvania for a while, and then was appointed to a position of responsibility with the Central Railroad of New Jersey. He held the latter position until 1874, when he was appointed Chief Engineer of the New York elevated roads, a position which he held for twenty-three years, when he retired.

William J. Johnston, publisher of the "American Exporter," died on Sunday, April 28, 1907, at his late residence, 774 West End Ave., New York City. He was born in Ballycastle, County Antrim, Ireland, in 1853, and educated there. He came to this country in 1868 and soon afterward became a telegraph operator in the office of the Western Union Telegraph Co. While in the employ of this company he conceived the idea of publishing a paper known as "The Operator," of interest to telegraph operators and people interested in electrical subjects. The Western Union Telegraph Co. was not in sympathy with the idea of one of its employees publishing a newspaper and the Superintendent, Mr. A. S. Brown, wrote Mr. Johnston as follows:

> It is my duty to inform you that your continued employment with us will necessitate your giving up all connection with the paper of which, I understand, you are now proprietor and publisher.

Although Mr. Johnston had practically no funds at this time his answer was as follows:

> You give me the choice of either giving up all connection with the paper, or leaving the service. I choose the latter and hereby tender my resignation, to take effect either immediately or on the first of January as you may think proper.

The "Operator" was later published as the "Electrical World" and is to-day the largest and most influential paper in the electrical field. Nearly all the men who were formerly connected with Mr. Johnston and who received their education in his office hold responsible positions to-day on technical papers throughout the country.

He disposed of the "Electrical World" in the late '90s and for a time published "The Engineering & Mining Journal," and afterward "The Mining Magazine."

Mr. Johnston was married twice; his first wife died about 19 years ago. His second wife was the daughter of J. H. Johnston, the well known jeweler, formerly of Union Square, West, New York. Besides the widow he leaves four sons and four daughters.

He was a member of the American Institute of Mining Engineers, an associate member of the American Institute of Electrical Engineers, and a member of the Hardware Club and the Sphinx Club.

ENGINEERING SOCIETIES.

COMING MEETINGS.

AMERICAN ELECTROCHEMICAL SOCIETY.
May 2-4. Annual meeting at Philadelphia, Pa. Secy., S. S. Sadtler, 39 South 10th St., Philadelphia, Pa.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
May 21-24. Annual convention at Philadelphia, Pa. Secy., Richard Moldenke, Watchung, N. J.

NATIONAL FIRE PROTECTION ASSOCIATION.
May 22-24. Annual Meeting at New York City. Secy., W. H. Merrill, 382 Ohio St., Chicago, Ill.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
May 28-31. Semi-annual Meeting at Indianapolis, Ind. Secy., Mr. Calvin W. Rice, Engineering Societies Bldg., 33 West 39th St., New York City.

NATIONAL ELECTRIC LIGHT ASSOCIATION.
June 4 to 7. Annual convention at Washington, D. C. Secy., W. C. L. Eglin, 29 West 39th St., New York City.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 12-14. Annual meeting at Atlantic City, N. J. Secy., J. W. Taylor, 390 Old Colony Bldg., Chicago, Ill.

MASTER CAR BUILDERS' ASSOCIATION.
June 17-19. Annual meeting at Atlantic City, N. J. Secy., J. W. Taylor, 390 Old Colony Bldg., Chicago, Ill.

AMERICAN WATER-WORKS ASSOCIATION.
June 17-22. Annual convention at Toronto, Ont. Secy., J. M. Diven, 14 George St., Charleston, S. C.

ASSOCIATION OF RAILWAY TELEGRAPH SUPERINTENDENTS.
June 19, 20. Annual meeting at Atlantic City, N. J. Secy., P. W. Drew, Wisconsin Central Ry., Milwaukee, Wis.

AMERICAN SOCIETY FOR TESTING MATERIALS.
June 20-22. Annual meeting at Atlantic City, N. J. Secy., Edgar Marburg, University of Pennsylvania, Philadelphia, Pa.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Annual convention at the City of Mexico, Mexico, beginning July 8. Secy., Charles Warren Hunt, 220 West 57th St., New York City.

ASSOCIATION OF TRANSPORTATION AND CAR ACCOUNTING OFFICERS.
June 25, 26. Annual meeting at St. Paul, Minn. Secy., G. P. Conard, 24 Park Place, New York, N. Y.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
June 25-28. Annual convention at Niagara Falls, N. Y. Secy., Ralph W. Pope, 29 West 39th St., New York City.

AMERICAN STREET AND INTERURBAN RAILWAY ASSOCIATION.
Oct. 14-18. Annual convention at Atlantic City, N. J. Secy., Bernard V. Swenson, 29 West 39th St., New York.

NEW YORK ELECTRICAL SOCIETY.—The 267th meeting was held at the power station of the Interborough Rapid Transit Company, 59th St. and 11th Ave., on Wednesday, April 24th. Over 500 of the members and friends of the society availed themselves of the opportunity for seeing the plant.

THE MUNICIPAL ENGINEERS' SOCIETY OF THE CITY OF NEW YORK held its regular monthly meeting in the Engineering Societies Building, on April 24, with President Rice in the Chair. The main paper of the evening was read by Mr. John T. Fetherston, Superintendent of Street Cleaning, Borough of Richmond, on "Some Notes on Refuse Destruction and Municipal Cleaning." The Borough of Richmond has under construction a new refuse destructor, in the study of the design of which Mr. Fetherston made an investigation of the various types of destructors which have been designed or built in this country and abroad. The paper consisted principally of a discussion of the qualifications needful in a successful destruction, together with illustrated descriptions of the various similar plants now in operation. Certain details of the street cleaning operation in the Borough of Richmond were also treated. Messrs. Stearns and Lewis followed the main paper with views of the operations of the street-cleaning department of the Borough of Manhattan. Mr. Stearns has designed several very interesting rubbish destructors and also dump piers, which he explained in some detail.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.—The regular meeting was held at the Engineering Societies Building, New York City, April 26, in two sections. In the auditorium, Mr. S. P. Grace, Chief Engineer of the Central District & Printing Telegraph Co., Pittsburg, Pa., presented a paper on "The Telephone Wire Plant." He stated that while the general impression among engineers was, that telephone engineering was largely a switchboard problem, this was not the real case. He stated that two-thirds of the total cost of a telephone plant was in the outside system. The method of planning the development of a telephone, with the greatest ultimate economy in construction and operation of the outside plant, was given for a city of 40,000 inhabitants. Standard practice was shown in detail, for conduits, feeders, aerial lines, and individual distribution.

In assembly room No. 2, on the fifth floor, Mr. D. McFarlan Moore, Chief Engineer of the Moore Electric Co., Newark, N. J., presented a paper on "Light from Gaseous Conductors Within Glass Tubes—the Moore Light," and gave a demonstration of the system. He stated that the problem he had sought to solve was a close imitation of daylight by vacuum tubes. The four lines of experiment for replenishing the gas, as made necessary by passage of current, have been as follows: (1) By catalytic action; (2) by evaporation and condensation; (3) by supplying the gas from the action of heat on chemicals; (4) by automatic feeding as necessary by an electrically operated valve, which is the present commercial system.

The assembly room was lighted by a Moore tube 1¾ ins. in diameter, arranged as a rectangle about the room 13 ins. from the ceiling. The tube had its ends in a terminal box containing a step-up transformer, electrodes and the vacuum regulating valve.

The test data presented indicated that the Moore system had about five times the illuminating efficiency of the incandescent carbon lamp under similar local conditions. Tubes fed with nitrogen give a yellowish light, claimed to be close to daylight. Those fed with carbon dioxide give a white light which requires a greater expenditure of energy to produce. To insure steady illumination, the power factor of a Moore tube is best at about 65%. Characteristic curves of the tubes were displayed. The low intrinsic brilliancy was shown by the equipment in the room. The stroboscopic effects were pronounced, however, and at first unpleasant.

Dr. C. P. Steinmetz led the discussion. He maintained that the efficiency tests of the Moore tube had been misinterpreted, and that its real efficiency placed it below the mercury vapor, and close to the tantalum lamps. He commended the perseverance of the inventor in bringing his first Geissler tube arrangement to the present commercial system.

Mr. Percy Thomas maintained that the light coming from an incandescent gas could not have a continuous spectrum as claimed. He called attention to lack of equal diffusion in all planes. The test work also, he claimed, was not accurate. Messrs. Gano S. Dunn, J. W. Howard, Leon Geyssler, M. Inst. E. E., and Dr. Sharp, of the Electrical Testing Laboratories, gave short discussions of other features.